LOUISIANA STATE UNIVERSITY
AND AGRICULTURAL AND MECHANICAL COLLEGE
Center for the Study of Public Health Impacts of Hurricanes

February 12, 2007

The Honorable Johnny Bradberry
Secretary, Louisiana Department of Transportation and Development
1201 Capitol Access Road
Baton Rouge, Louisiana 70804-9245

RE: Transmittal of 'Team Louisiana' Final Report: *The Failure of the New Orleans Levee System during Hurricane Katrina*

Dear Secretary Bradberry:

We, the members of "Team Louisiana", wish to express our appreciation for your foresight and the trust you placed in us at a time when the causes of failure were quickly being obscured in the rush to make repairs. Please accept the accompanying final report documenting our findings. As per the contract, we did not include recommendations in the body of the report. We include them here in the transmittal letter for your reference.

The impacts of the surge generated by Hurricane Katrina, a Saffir-Simpson Category 3 storm on landfall, are unprecedented in U.S. history. There is a potential for any forensics investigation to convey an apparent omniscience derived from 20:20 hindsight, and to lose sight of key points like this one. Team Louisiana tried to avoid this trap by focusing on what was known at the time the Greater New Orleans Hurricane Protection System (GNO HPS) was designed, the analytical tools that were available then, and what tools were used.

On the other hand, engineers – then or now – all work with uncertainty and follow accepted practice to account for unknowns that increase the risk of failure. As one of the senior Team Louisiana geotechnical engineers pointed out, it is the anomalous stratum, rather than the average soils condition, that generally causes foundation failure. Engineers address these uncertainties in levee and floodwall design by adding freeboard to raise crown elevation beyond the minimum specified, by inflating the stress to be resisted by a "factor of safety" sufficient to account for unknowns, and by incorporating redundant measures to limit the effect of the failure of a single component. These are some of the key features that distinguish a safe system from one that is unsafe. Such elements are the focus of this investigation.

The following key questions were formulated to guide the forensics investigation.

1. Was the GNO HPS properly conceived to accomplish the 1965 Congressional mandate to protect against the "most severe combination of meteorological conditions reasonably expected?"

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2. Were the levels of protection, or crown elevations, specified in designs for HPS elements sufficient to resist overtopping by surge and waves associated with the 100-year Standard Project Hurricane?
3. Did incorrect design assumptions compromise performance? Should these have been detected and corrected by engineers equipped with the tools available at the time?
4. Did the Mississippi River Gulf Outlet (MRGO), a free-flowing, deep-draft navigation canal that pierced the HPS on the eastern side, compromise system performance?
5. Was the system maintained and operated to assure the required level of protection through time? Specifically, how did the 40-year construction schedule impact system performance?

These questions are each answered in a separate chapter of the report, with a brief recapitulation in the executive summary. The following are recommendations that were developed by the team as an outgrowth of our participation in the forensics effort.

<u>Independent Review of All Levee Systems and Protection Projects</u>

- Katrina was a largely man-made catastrophe triggered by a moderately fast Category 3 hurricane that missed New Orleans. Citizens were denied the level of protection mandated by Congress in the 1965 Flood Control Act. Accordingly, we recommend that the US Congress empanel independent review teams, with substantial participation by Louisiana scientists and engineers, to assess the integrity and safety of all existing federal and state levee systems in Louisiana, including those currently under repair. Similarly, we recommend that the state – perhaps through its newly consolidated levee boards -- carefully assess the safety and reliability of all proposed federal projects that could affect public safety or the functioning of ecosystems, particularly as they will be affected by sea level rise and subsidence. The Louisiana Legislature should ensure ongoing funding for this purpose. Interactive consultation and review by consulting panels of leading outside experts is common practice in dam engineering.
- Based on the failures studied by Team Louisiana, no cantilevered sheet pile structures nor "I-walls" in southeastern Louisiana should be certified to sustain hurricane loadings without careful inspection and analysis of the geotechnical data used in the design, by the independent review team mentioned above. All I-walls should be re-evaluated for current design loadings assuming a water filled gap along the flood side of the wall. All levees underlain by soft clay or other such questionable soils should be re-evaluated for current design loadings accounting for reduced shear strength of soils in areas at or beyond the toe of the levee. Any walls found structurally sound should be provided with splash pads to prevent failure under overtopping conditions. It should not simply be assumed that pile-supported 'T-wall' systems are safe.
- Louisiana should take the lead in determining the level of hurricane protection that should be provided to citizens of the state. This is a policy decision that should not be left to federal agencies or outside experts. A risk-based approach is better than the obsolete "Project Storm" approach, but it would be even more useful to strive for a particular public safety or policy goal, like protection for New Orleans at a level that

renders evacuation undesirable. The Louisiana Legislature should empower and fund Louisiana Department of Transport and Development to undertake this critical task.

8-29 Commission/Levee Commission

- All of the existing forensics reports, including ours, raise technical questions about the way in which the USACE designs and builds projects with critical life-support functions, but do not tell us why decisions were made in the way they were, and therefore cannot build confidence in the USACE going forward.
- Transparency in dealing with the public is a must. Citizens of New Orleans were never told by those with both knowledge and responsibility just how vulnerable they were to flooding, or of the public safety compromises made in designing and building structures. It is extremely important that the capacity of the existing protection system be fully communicated to the public, as well as the public safety consequences of upgrades and repairs. They need to understand the risks for living behind the federal levee system.
- Because trust is more difficult to rebuild than levees following catastrophic failure, we recommend that the issues raised in the forensics portion of the Team Louisiana and Independent Levee Investigation Team (ILIT) reports form the basis for an official inquiry, by US Congress, into how mistakes with such tragic consequences were made.
- The unique blanket immunity that the USACE enjoys in the law from liability for defective design of flood control projects should be re-examined, by the US Congress, and carefully restricted to ensure lines of corporate and professional responsibility are clear and traceable.
- An official inquiry along the lines of the "truth and reconciliation" commissions that have followed political turmoil in other countries would seek not to brand scapegoats but to encourage those who with special knowledge to explain what they know without fear of retribution to ensure that the same mistakes are not made again. The goal is to allow both those harmed and those who were responsible to move beyond the tragedy with rebuilt respect and trust into a very different future.
- Following on the 8-29 Commission/Levee Commission, the US Congress should pass a "Katrina Recovery Bill" that will ensure that coastal restoration and protection are fully federally funded, and New Orleans' recovery supported over the long-term, in recognition of the role played by federal decisions in contributing to the losses.

Integration of Navigation, Coastal Protection and Restoration Projects

- Unlike a river flood that may last for months, the challenge presented by a hurricane surge lasts only a matter of hours at any one point. An initial Master Plan is being conceptualized by the State through the Coastal Protection and Restoration Authority (CPRA). We recommend that the State invest in a more rigorous effort to go beyond concepts laid out in the CPRA plan to integrate coastal restoration and protection projects in ways that will accomplish important policy goals, such as eliminating the need to evacuate New Orleans, for example. The current draft CPRA Master Plan does include recognition of the surge and wave reduction potential of natural landscape features like wetlands and barrier islands, but does not specifically incorporate many of the lessons of Katrina and Rita with regard to the reliability of engineered structures.

- All existing federal navigation projects in coastal areas should be evaluated by independent investigators using modern surge and wave modeling techniques to determine the degree to which they continue to compromise existing storm flood protection. This action should be fully funded by the US Congress, not undertaken by the USACE, and the state must have a significant role. This action can form the basis for Federal Emergency Management Agency (FEMA) remedial retrofit plans. The MRGO and other channels in the funnel must immediately be fitted with appropriate closure features by the USACE to ensure that they do not serve again as a conduit for surge and waves to Greater New Orleans.

Relationship between Local Sponsors and the Federal Government

- Local levee boards and other sponsors in the Greater New Orleans area kept levees and rights of way mowed, operated drainage structures, commented on USACE design memoranda, and participated in inspections. They were not, however, consulted on critical design or construction decisions. On the other hand, they had to pay 30 percent of all costs incurred for a level of protection that appeared on some reaches to diminish over time. The USACE was unresponsive to critiques coming in from cost-sharing partners and was often ready to pass incomplete project elements to local authorities without local concurrence. The sponsors were expected to rubber stamp USACE design documents. Only by withholding cost-sharing funds could these partners bring attention to technical concerns, and this tended to make every technical issue into a political one. The one-sided relationship between local sponsors and the USACE on technical management and oversight of cost-shared projects should be changed by the US Congress to create an atmosphere in which technical inquiries are encouraged and given respectful treatment outside of a political arena.
- We recommend that all levee certifications necessary for local government participation in the federal Flood Insurance Program be renewed on 5-year or 10-year intervals based on data that is stamped by professional engineers and available for public inspection, rather than a simple blanket assertion by the USACE.
- The State should establish clearer lines of responsibility when it comes to the operation and maintenance of the drainage systems and flood control systems. One problem is that the federal projects are never completed, rendering a clean hand-off to local authorities impossible. It is hoped that the consolidated levee boards established by statewide ballot in Louisiana in 2006 will improve coordination.

A Solution to the Settling of Levee Structures

- Settling of levees and floodwalls is a recognized factor affecting the reliability of these features. We recommend that the State and USACE adopt a life-cycle approach to comparing alternatives for critical flood control structures that gives more weight to on-time completion and reliability. Because of settlement and subsidence, it is essentially impossible to ever finish construction of gravity supported levees, or floodwalls that depend on levees for support (e.g. I-walls), in the GNO area. Actual crown elevations for these structures describe a trajectory over time between enlargements such that the actual level of protection rarely corresponds to the design level. In the interest of completing projects on a reasonable schedule, the State and USACE should consider using pile-supported protective structures to a much greater degree than in the past even if the first

construction cost is much higher. Construction lifts could conceivably be limited with deep pile supported floodwalls and barriers.

The infrastructure failures observed in the New Orleans Area and described in the accompanying Team Louisiana report resulted in tragic loss of life and devastating property damage that continues to threaten the viability of the State's largest city. The failures were foreseeable, for the most part, and could have been prevented at a fraction of what it has cost to repair the current system. Although it has been a somber task, we have appreciated the opportunity that you provided for us to dig deeply into the details of what went wrong. There are many hard-earned lessons, and we have provided the recommendations listed above in the hope that the mistakes we make in the future will at least be different, and that we have done all that we can to create a more reliable system for our children and grandchildren.

Sincerely,

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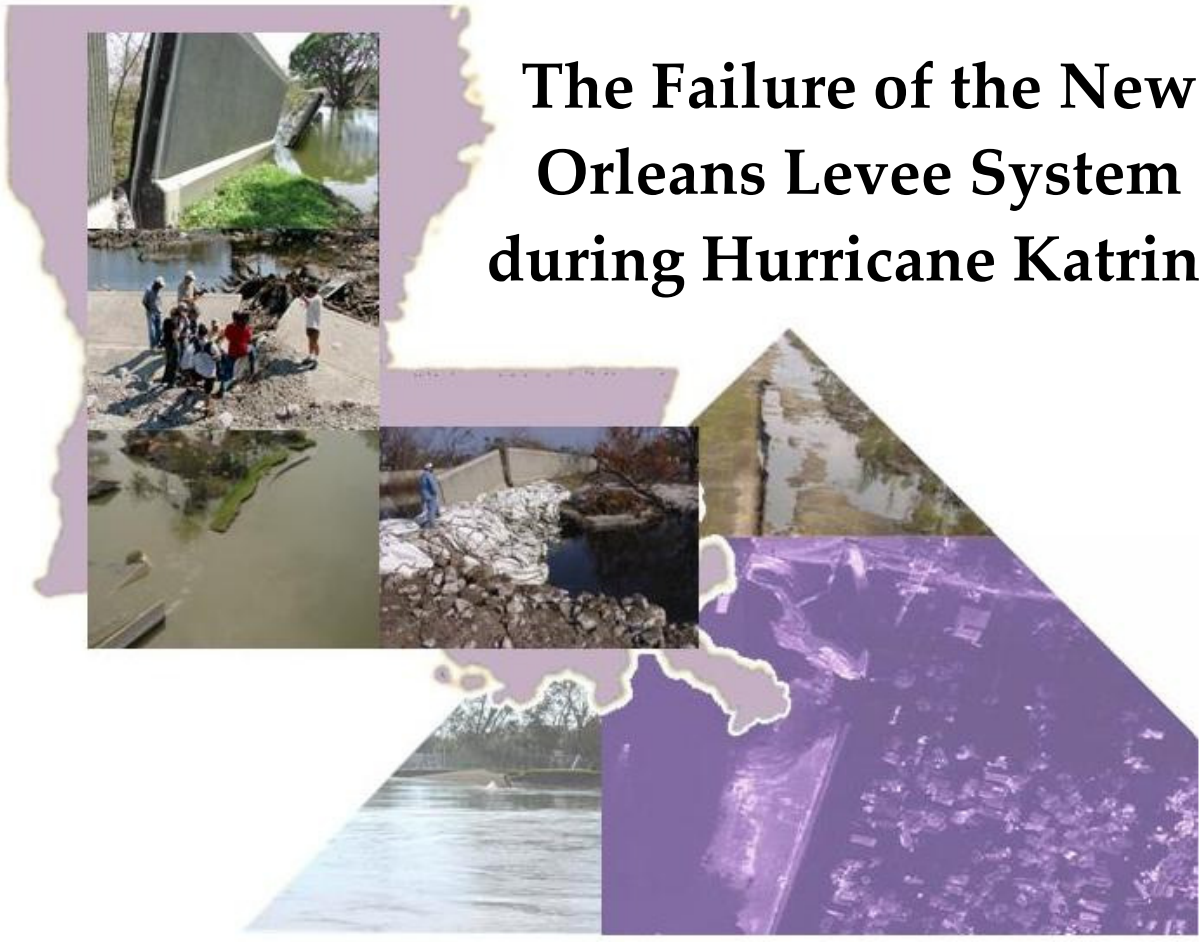

The Failure of the New Orleans Levee System during Hurricane Katrina

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Center for the Study of PUBLIC HEALTH IMPACTS of Hurricanes

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LSU HURRICANE CENTER
Addressing Hurricanes and Other Hazards and Their Impacts on the Natural, Built and Human Environments

A Report prepared for Secretary Johnny Bradberry
Louisiana Department of Transportation and Development, Baton Rouge, Louisiana
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ACKNOWLEDGEMENTS

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Executive Summary

Louisiana State University (LSU) was commissioned in October, 2005 by the Louisiana Department of Transportation and Development (LDOTD) to assemble a team of Louisiana-based academic and private sector experts to "collect forensic data related to the failure of the levee systems around greater New Orleans" that occurred during passage of Hurricane Katrina on the morning of 29 August 2005. This group, later known as 'Team Louisiana,' was to focus on the hurricane protection system (HPS) designed and constructed over a 40-year period by the U.S. Army Corps of Engineers (USACE) for the East Bank of the Greater New Orleans area (GNO), including New Orleans East and St. Bernard Parish.

One way to look at the Katrina event is as a catastrophic natural disaster, and, with respect to the magnitude of the storm surge, it was. This approach tends, however, to minimize the engineering contribution to the direct or indirect loss of as many as 1,500 Louisiana residents (including the over 130 still missing as of December 2006, most considered swept away and drowned). Over 100,000 families were rendered homeless, making the destruction of New Orleans the worst from that perspective since the record Mississippi River flood of 1927. The response of the Nation to that natural disaster, even though it cost far fewer human lives, came in the form of an unprecedented engineering program to ensure that the flooding of the Lower Mississippi Valley would never happen again. The federal HPS that was authorized in 1965 to protect New Orleans following Hurricane Betsy had the same goal, but was clearly ineffective. It is important to understand why.

From an engineering perspective, forensics science is the study of materials, products, structures or components that do not operate as intended. In the context of the flooding of New Orleans, the purpose is to understand, first, what performance was expected from the GNO HPS and, second, to identify causes of failure as part of an effort to improve future performance.

Team Louisiana was asked, more specifically, to develop a time-history of surge and wave elevations for levee and floodwall reaches that failed, to compare this information to the designed and actual levee crown and floodwall crest elevations, and to assemble and examine all relevant design memoranda, construction plans and as-built surveys. In addition, Team Louisiana was asked to participate in debriefing of eye-witnesses, assembling stopped clock data, collecting aerial and ground level photographic evidence, and conducting non-destructive testing to determine soil foundation conditions and sheetpile depths in the vicinity of floodwall breaches.

At the time that Team Louisiana was commissioned, researchers from the LSU Hurricane Center had already fielded a reconnaissance effort that had uncovered apparent discrepancies between what was observed and early USACE statements about the causes of levee and floodwall failures. Following public discussion of these findings, three other investigations were organized by external groups as diverse as the University of California, Berkeley and the American Society of Civil Engineers, as well as by the USACE itself. These investigative teams included few scientists or engineers from Louisiana. Secretary Johnny Bradberry of the LDOTD saw a need for an official state-sponsored initiative to ensure that state and local perspectives were not ignored as the investigations proceeded.

The external study teams, particularly the Independent Levee Investigation Team (ILIT) that grew out of the UC Berkeley initiative, and the Interagency Performance Evaluation Team (IPET) sponsored by the USACE, concluded their work and issued final reports earlier this summer. The findings of these investigations differ with respect to some details, but generally concur on the specific mechanisms of most of the foundation-related floodwall failures. It is not surprising, however, that from a local perspective, these external probes appeared to miss some of the context in which the design and construction of the still incomplete federal HPS – originally planned to take 13 years at a total cost of less than $90 million – stretched out over 40 years.

The GNO HPS project employed two generations of USACE employees at an estimated total cost more than $700 million, while consuming nearly $200 million in locally generated funds. Over this time, the USACE provided local sponsors with many conflicting claims, but few reliable assurances, of the actual level of protection being provided. The cost to repair the GNO HPS to pre-storm condition has cost as much in the year since Katrina as was spent in the previous 40 years. The repaired HPS still provides a substantially lower level of protection than was originally authorized in 1965. Given this history, the multi-generational tension between the USACE and those being protected in the GNO is complex and easily misunderstood. It is hoped that this report will enrich the historical record and provide additional local perspective.

Dr. Ivor van Heerden, Director of the Center for the Study of Public Health Impacts of Hurricanes and Deputy Director of the LSU Hurricane Center, was selected to lead these efforts. Dr. van Heerden recruited three other LSU scientists including an oceanographer, a hydraulic engineer, and a geotechnical engineer. This group of academic researchers was significantly augmented by the addition of three senior engineers from the private sector. These members included two geotechnical

engineers and a water resources engineer who had each participated in the design of numerous flood control works in the GNO area across the entire 40 year evolution of the Lake Pontchartrain and Vicinity Hurricane Protection Project.

The surge generated by Hurricane Katrina, a Saffir-Simpson Category 3 storm on landfall, is unprecedented in U.S. history. There is a potential for any forensics investigation to convey an apparent omniscience derived from 20:20 hindsight, and to lose sight of key points like this one. We have tried to avoid this trap by focusing on what was known at the time the GNO HPS was designed, the analytical tools that were available then, and what tools were used.

On the other hand, engineers – then or now – all work with uncertainty and follow accepted practice to account for unknowns that increase the risk of failure. As one of our senior engineers pointed out, it is the anomalous stratum, rather than the average soils condition, that generally causes foundation failure. Engineers address these uncertainties in levee and floodwall design by adding freeboard to raise crown elevation beyond the minimum specified, by inflating the stress to be resisted by a "factor of safety" sufficient to account for unknowns, and by incorporating redundant measures to limit the effect of the failure of a single component. These are some of the key features that distinguish a safe system from one that is unsafe. Such elements are the focus of this investigation.

This report is organized in two parts. The first part provides background information critical to understanding the physical and historical setting relevant to flood protection and drainage; the magnitude and sequence of stresses placed on the HPS by Hurricane Katrina; and the nature of the flooding that directly or indirectly led to the loss of as many as fifteen hundred Louisiana residents, and the destruction of much of New Orleans. The second part addresses the forensics issues as we see them. The following key questions were formulated to guide the forensics investigation. Each is discussed briefly here, and examined in more detail in a separate section of this report.

1. Was the GNO HPS properly conceived to accomplish the 1965 Congressional mandate to protect against the "most severe combination of meteorological conditions reasonably expected?"
2. Were the levels of protection, or crown elevations, specified in designs for HPS elements sufficient to resist overtopping by surge and waves associated with the 100-year Standard Project Hurricane?

3. Did incorrect design assumptions compromise performance? Should these have been detected and corrected by engineers equipped with the tools available at the time?
4. Did the Mississippi River Gulf Outlet (MRGO), a free-flowing, deep-draft navigation canal that pierced the HPS on the eastern side, compromise system performance?
5. Was the system maintained and operated to assure the required level of protection through time? Specifically, how did the 40-year construction schedule impact system performance?

Question 1. Was the GNO HPS properly conceived to accomplish the 1965 Congressional mandate to protect against the "most severe combination of meteorological conditions reasonably expected?"

Answer 1. No. The initial meteorological and oceanographic analysis based on the 1959 U.S. Weather Bureau 1 in 100 year Standard Project Hurricane (SPH) was known to be obsolete by 1972, just as construction of initial parts of the GNO HPS was getting underway. The primary deficiency of the 1959 SPH was in the specification of maximum sustained wind speed, which the National Weather Service (NWS) had increased by 20 percent, from 107 to 129 mph. The steady-state analytical approach used by the USACE to develop surge estimates was as sensitive to the effect of wind velocity as later numerical modeling approaches (i.e. SLOSH or ADCIRC), and should have alerted the USACE to the danger of underestimating wind speed. This analysis provided a design basis for setting the minimum heights above mean sea level for levee and floodwall crowns to resist overtopping by combined SPH waves and surge. A 20 percent underestimate of maximum winds can lead to a 40 percent reduction in the predicted surge elevation. In 1979 the NWS raised the maximum sustained winds to 140 MPH, a category 4 hurricane!

The New Orleans District USACE was aware of this deficiency in the original analysis, as is indicted by testimony in 1976 and 1982 General Accounting Office (GAO) reports, but never revised the original SPH-based analysis to reflect the new understanding of the threats, even after being ordered to do so by the Chief of Engineers in 1981 (ER 1110-2-1453). New Orleans residents were not advised that the GNO HPS required significant improvements to meet 1 in 100 year SPH requirements, but, instead, the New Orleans District claimed at times that the GNO HPS would protect against a 1 in 200 to 1 in 300 year hurricane. No basis for this claim has been established, while numerous storms that have affected the GNO area – before and after the 1965 initiation of the HPS -- were more severe than the 1959 SPH.

The New Orleans District (NOD) USACE missed opportunities to revise the original SPH-based analysis after the NWS revised the SPH in 1972 and 1979, and when the SLOSH storm surge model came into use in 1979. SLOSH showed clearly that the GNO HPS, as it was constructed at the time, was vulnerable to overtopping by many possible Category 3 storms. This result was confirmed later by the ADCIRC model, as recently as during the 2004 FEMA Hurricane Pam exercise (http://hurricane.lsu.edu/floodprediction/PAM_Exercise04/). The USACE supported development of both surge models and was aware of GNO HPS vulnerabilities, but appeared to accept the inadequacy of the system with a complacency that undercut efforts to sound alarms and begin pressing for improvement.

Question 2. Were the levees and floodwalls at or above the crown elevations specified in designs for HPS elements necessary to resist overtopping by surge and waves associated with the Standard Project Hurricane?

Answer 2. No. Floodwall and levee crown elevations were built 1 to 2 ft low because of an erroneous assumption at USACE New Orleans District (NOD) that an elevation of zero referenced to the National Geodetic Vertical Datum of 1929 (NGVD29) was equal to -- and interchangeable with -- local mean sea level (LMSL). LMSL was the relevant datum for superimposition of hurricane surge and wave height from a 1950's era oceanographic analysis. In 1965, zero NGVD29 was between 1.3 and 1.6 feet below LMSL at different parts of the system, and floodwalls and levee crowns were constructed lower by this margin. This mistake was locked in for continuing HPS construction when the NOD adopted a policy in 1985, with the approval of the USACE Lower Mississippi Valley Division (LMVD), to explicitly use the outdated 1965 NGVD29 adjustment for elevation control. As a result, no provision was made to account for the 3 to 4 ft/century subsidence rates characteristic of the GNO area even though this rate was known at the time of authorization. Crown elevation deficiencies ranging up to 5 feet at the time Katrina struck resulted in prolonged overtopping of floodwalls and levees along the Inner Harbor Navigation Canal (IHNC) and to the east in the Lake Borgne funnel that otherwise would have been overtopped only briefly. Prolonged overtopping led to catastrophic breaches into the Lower 9th Ward on the east and into Orleans Metro on the west, and contributed to the early failures of levees along the Gulf Intracoastal Waterway (GIWW) and MRGO. Early failure of the MRGO levee allowed the 32,000 acre wetland buffer between MRGO and 40 Arpent back levee to fill and overtop the 40 Arpent back levee while the surge was still rising, and resulted in catastrophic flooding in St. Bernard to an elevation of 11 ft (NAVD88).

Question 3. Did the USACE follow existing engineering practice and USACE guidance for construction of levees and floodwalls? Should issues about levee

materials and floodwall designs have been detected and corrected by engineers equipped with the tools available at the time of construction?

Answer 3. No to the first question, and yes to the second. Weak soil strengths or potential for underseepage were evident in strata tested for the USACE during the early 1980s under Orleans Metro drainage canal floodwall levees that failed. The potential consequences of these layers on levee stability were known to practicing engineers at the time but were missed or ignored because of inappropriate averaging of soil strengths on long levee reaches and across layers. Design engineers assumed that consolidation of soils beneath the I-wall levees on the 17th Street Canal would have increased soil strengths over time, but borings and soundings conducted since Katrina show that very soft clays in the failure zone have strengths less than values assigned in 1981. Where Division-level reviewers identified potential problems, they were rebuffed by District personnel citing "professional judgment."

The New Orleans District USACE failed to conduct appropriate analyses of the potential for seepage to compromise levee and floodwall stability where shallow sand deposits occurred beneath the levee, such as at the London Avenue Canal. Design memoranda indicate reliance upon the Lane's Weighted Average Creep method for underseepage analysis. This method was recognized in the profession at the time to be inappropriate for final design in a critical life-support structure. The presence of layered sands and clays should have led to analysis using more rigorous flow net and finite-element techniques in widespread use at the time, and specified in the governing USACE engineering manual for Design and Construction of Levees (EM 1110-2-1913, 1978 ed.). Sheetpile supported I-walls that were installed on levees with cross-sections too small to prevent underseepage also did not provide sufficient resistance when fully loaded, no matter what the sheetpile length. There is no evidence that rigorous analysis of uplift pressures was undertaken.

Idealized design templates were applied to long levee and floodwall reaches without adjustment for variable subsoil conditions or for variations in elevation on the protected side. IPET believes that such a mistake caused the levee supporting the I-wall levee to be constructed improperly in the vicinity of the north breach into the Lower 9th Ward, where the ground elevation on the protected side was lower. The foundation may have failed early in the storm sequence at a water elevation well below the design level of protection because of inadequate resistance.

The New Orleans District USACE did not follow standard engineering practice or Corps guidance when evaluating whether to protect (armor) earthen sea dikes from erosion caused by waves in the funnel area east of the city. Such evaluations should

have followed the 1954 TR-4 Shore Protection Planning and Design (Beach Erosion Board) or its successor, the Shore Protection Manual, first published in 1973. Instead of the required analysis, Design Memoranda for the New Orleans East and Chalmette Levees substitute the following disclaimer.

> "Due to the short duration of hurricane flood stages and the resistant nature of clayey soils, no erosion protection is considered necessary on the levee slopes."

These levees were not designed to withstand general overtopping, as was amply demonstrated in Katrina, but were expected to experience overtopping by waves greater than the significant wave provided in the oceanographic analysis. Many miles of the Chalmette and New Orleans East Levees were constructed of shell-rich sands with poor erosion resistance derived from the hydraulic excavation of the adjacent GIWW and MRGO channels, rather than the hauled clay soils specified for levees protecting urban areas (EM 1110-2-1913, 1978 ed.).

Question 4. Did the free-flowing, deep-draft navigation canal that pierces the HPS on its eastern side compromise system performance?

Answer 4. Yes. The MRGO and GIWW channels provide efficient conduits to funnel surge into the heart of New Orleans. As a result, surge elevations peaked in Lake Borgne and the IHNC almost simultaneously at higher levels relative to levee and floodwall crowns, and earlier, than would have been true if the MRGO had not been built, and if the wetland loss it caused had not occurred. The effect of these federally constructed and operated channels on surge and waves has consistently been underestimated by the USACE from before Hurricane Betsy, right through to the recent IPET report, as has the effect of accelerated wetland loss in the funnel area. One consequence of this institutional "blind spot" was that a hurricane barrier of the type proposed in the original pre-1980s HPS for the other two main passes into Lake Pontchartrain was never included for the MRGO.

The ILIT and IPET have indicated that the original "barrier" approach was a better design than the "high-level," levees-only HPS ultimately adopted twenty years after authorization. But our work indicates that disastrous flooding during Katrina from the Lake Borgne funnel and the IHNC would have been exacerbated by the barrier proposed at the Lake Pontchartrain terminus of the IHNC (Seabrook). On the other hand, the Lake Pontchartrain surge along the south shore might have been reduced by up to 3 ft, and by a greater margin on the north shore, by the barriers that were proposed for the two other Lake Pontchartrain passes. This might have been enough to

prevent one or more of the failures of the defective Orleans Metro drainage canal floodwalls built in the 1990s, and this would have greatly reduced the severity of prolonged flooding in Orleans Metro. Such trade-offs were never rigorously assessed when the decision was made to change the HPS design in such a major way in 1985 at a time when surge modeling techniques using SLOSH were available. Again, the level of protection was reduced without informing the population at risk.

Question 5. Was the system maintained and operated to assure the required level of protection through time? Specifically, how did the 40-year construction schedule impact system performance?

Answer 5. No. The GNO HPS was managed like a circa 1965 flood control museum. Design assumptions and policy made in 1965 continue to diminish the HPS today. Local sea level has risen 0.4 ft since the 1960s and much of New Orleans has sunk over 1.5 ft in the same period for a combined change of nearly 2 ft relative to sea level, but as IPET (II-78) noted,

> "It was not clear how projected subsidence rates were applied in structural elevation design, if at all. Subsidence was apparently not factored into the design freeboard allowance."

Prudent engineers operating in coastal Louisiana have made allowances for subsidence for a century. The New Orleans District was one of the first agencies to directly map coastal wetland loss in Louisiana, but this ever continuing diminishment of surge protection was never incorporated into design philosophies. An analysis of all factors affecting levee elevation is required as part of FEMA Levee Elevation and Certification Requirements (44CFR65.10). It is inexcusable that this was not done for what was the most critical urban coastal protection project in the country.

Most public works structures would be scheduled for replacement or rehabilitation after 40 years, but planning for a more modern system was put off while the original project fell farther and farther behind. Because the USACE never completed the 1965 project, it could not legally pass responsibility for major maintenance or upgrades to the local sponsors, or initiate a new project to bring protection to a higher standard. Local sponsors kept levees and rights of way mowed, operated drainage structures, commented on USACE design memoranda, and participated in inspections. They were not, however, consulted on design or construction decisions. On the other hand, they were required to pay 30 percent of all costs incurred for a level of protection that appeared on some reaches to diminish over time. When Katrina struck, the crown height on most levee and floodwall reaches was between 1 and 3 ft low relative to

current mean sea level, the only datum that is relevant to the oceanography of hurricane waves and surge. On the IHNC and MRGO, every foot of crown deficiency when the surge was above 11 ft meant that overtopping and levee erosion started a half hour earlier. A half-hour in the lifetime of a moving hurricane can mean the difference between success and failure.

Lessons of Forensics Findings

The design assumptions for the GNO HPS remained static despite growing scientific evidence that the threat posed by surge was actually far greater than originally estimated. Design histories for individual HPS elements showed a pervasive trend over time toward substitution of less reliable structures for more conservative designs. On a regional perspective, management of the Lake Pontchartrain and Vicinity project led, over time, to ever greater departures from the overall objective of the original authorization, to protect against the ***"the most severe meteorological conditions considered reasonably characteristic for that region."*** This Congressional objective could be read as a mandate for continual reevaluation and adjustment.

In fact, the project did evolve in a different way, as it grew to incorporate greater undeveloped areas to the east and west of the City and as the original barrier plan was abandoned in favor of the "high-level" levees-only alternative. So, the original project was changed in fundamental ways without rigorous analysis of the trade-offs in level of protection afforded and reliability. The 1984 Reevaluation, inspired by court decisions questioning whether the environmental impacts of the proposed Lake Pontchartrain barriers had been adequately assessed, could have been seen as an opportunity to critically examine the overall integrity and reliability of the system, and to improve the design (USACE 1984). Instead, the USACE concluded that it would cost more to leave out undeveloped wetlands than to continue the construction already started on the longer routes initially chosen to aid private drainage and development efforts.

In contrast to this history of expansion and modification, other decisions were made within the USACE organization presumably to control costs that were rationalized as a meticulous adherence to the largely discarded 1965 authorization. Contractors were required to use outdated benchmark elevations during construction, for example, while the erroneous substitution of the NGVD29 datum for the mean sea level datum specified in the original SPH oceanographic analysis – like that analysis itself -- never got a second look.

Role of Local Governments, Levee Boards and the State

Interactions between the USACE and local cooperating or sponsoring agencies affected the design of protective structures like the parallel floodwalls along the banks of the

Orleans Metro drainage canals. Although the federal government had overall responsibility for the GNO HPS, the slow pace at which federal funds were made available ($3 to 5 million per year) led local agencies and their contractors to take a lead in many cases to get work started with local funds. As has been discussed, the USACE escalated the protection claimed for a completed Lake Pontchartrain and Vicinity Project from the 100-year to the 300-year storm level without changing any proposed structures. This claim led local engineers to believe that designs originally proposed for some HPS elements were excessively conservative, and that an adequate system could be constructed more quickly and at lower cost without significantly sacrificing performance or reliability. In contrast, most investigators who have reviewed the designs after Katrina have concluded that factors of safety applied by the USACE were anything but conservative given the criticality of subsurface soils and the consequences of failure.

Levee districts are state commissioned entities advised on engineering issues by the LDOTD. LDOTD assumed this function after a reorganization in which it absorbed the duties of the earlier Louisiana Department of Public Works in 1978. The levee districts have the authority to raise funds within their boundaries for flood protection projects. They have typically been the cost-sharing sponsors for development of jointly funded federal hurricane protection systems. They also assumed limited responsibilities for maintenance of portions of the federal system that the USACE decided were "substantially" complete, if not actually finished.

Another important factor came into play, however, after the National Flood Insurance Program (FIP) was established in 1968. Local governments sought to enhance economic growth by encouraging residential and commercial construction in new areas, often former wetlands, which were ringed by relatively low levees and subject to pumped drainage. With the advent of the FIP, development in these newly drained areas could proceed only if those who purchased properties there could also protect that investment with federal flood insurance. The Federal Emergency Management Agency (FEMA) would permit new areas to enter the FIP only if the levees and drainage system could be certified as providing protection against both the 100-year storm surge and 100-year rainfall event.

In the GNO, FEMA relied upon the USACE to provide engineering evaluations of flood risk in areas protected by both federal and non-federal levee systems. So local governments, rather than the levee districts, entered into discussions with the USACE to find out what minimal levee heights were necessary for certification against the 100-year surge event. If the USACE found that the perimeter levee system was high enough to prevent overtopping, then FEMA generally accepted this finding without requiring

geotechnical or construction information normally called for to meet FEMA levee standards (44CFR65.10).

Once the USACE determined that the levees were adequate, or, quite often, slated to become adequate sometime in the future, the "protected" areas were then analyzed only for the capacity of the internal drainage system to remove rainfall. The probability that the perimeter levee would be breached or overtopped was not considered. The internal drainage capacity was then used to determine the Base Flood Elevation (BFE) that governed how high buildings had to be elevated, and the location of flood zones on the Flood Insurance Rate Maps (FIRMs). Development accelerated once BFEs and FIRMs were issued. Thousands of people moved into suburban areas on the outskirts of the GNO in some cases before levee and drainage systems were complete or fully functional.

The IPET found that the GNO HPS failed to function as a system, but from a local perspective the HPS was successful for decades as a multi-purpose economic development tool that had an important role in facilitating drainage. Extreme rainfall events were far more frequent than hurricane surges, and apparently could be addressed, at least in the short-term, without a complete SPH-level hurricane protection system in place. Local officials were all too ready to believe glib assurances from the Nation's premier civil engineering organization that they were well protected against 100-year hurricane flooding.

Where Things Stand Today

The Lake Pontchartrain and Vicinity project employed generations of USACE employees and contractors at an estimated total cost of more than $500 million, while consuming nearly $200 million in state and locally generated funds. Prior to Katrina, it was estimated to be about 85 percent constructed, but was not expected to reach completion until 2015. A similar West Bank and Vicinity project had been initiated on the other side of the river in 1986 after serious flooding associated with Hurricane Juan. The West Bank HPS was expected to cost $330 million, again with a 35 percent local cost share, and was only 38 percent built prior to Katrina. Though it started much later, it was scheduled to be completed only a year after the east bank HPS. This much less capable system was not tested in 2005 to the same degree as the east bank HPS.

The pre-Katrina combined estimate of cost to complete the east and west bank GNO hurricane protection systems, a total of about $1 billion, can be compared with costs recently compiled by the USACE for emergency repairs to the two projects since Katrina, and with estimates of additional expenditures necessary to achieve a more realistic 100-year level of protection. Emergency repairs carried out by USACE

contractors in the year since Katrina to return the GNO levees and floodwalls to the pre-storm condition have cost between $400 and $600 million, if the interim lakeshore drainage canal closures are included.

The repaired HPS still provides a substantially lower level of protection than that originally authorized in 1965. Dr. Bob Bea, Co-Director of the University of California, Berkeley, Center for Catastrophic Risk Management, and an ILIT member, recently pointed out that "the repaired sections of the hurricane protection system are the strongest parts," but that "strong pieces embedded within weak pieces do not translate to a reliable system" (Bea 2006). Currently authorized projects to construct permanent lakeshore closures with pumps for Orleans Metro drainage canals will add an additional $100 to $200 million. Surge gates for the IHNC and MRGO are expected to cost at least $200 million more.

It is evidence of how pervasively under-built the system was that it has cost as much after Katrina to repair the GNO HPS to a marginally stable pre-storm condition as was spent in the previous 40 years. The USACE now estimates that between $2 and $4 billion will actually be required to achieve the minimal 100-year level of protection generally required for participation in the Federal Emergency Management Agency (FEMA) flood insurance program. The level of protection that this will achieve should not be confused with the much higher Category 5 hurricane protection now being studied by the State and USACE. That will cost much, much more.

The Barrier Plan Revisited

One of the flaws of the Lake Pontchartrain and Vicinity Project authorized in 1965 is inherent in its name. The primary threat to New Orleans at the time the oceanographic analysis was being conducted in the 1950s -- prior to the construction of the MRGO and before most of the suburban development in New Orleans East and St. Bernard -- was always seen as coming from the Lake. This is apparent in the planned Seabrook structure, which, had it been in place during Katrina, would have been as useful as the French Maginot Line in World War II, addressing a threat coming from the wrong direction. The highest storm surges that have caused flooding in the GNO since Hurricane Betsy have always come from Lake Borgne rather than Lake Pontchartrain, and this is likely to remain the most probable scenario going forward.

Today, the USACE seeks funds to rebuild flood defenses for a ruined city that will offer a level of protection originally conceived in the late 1950s. The evolution of the Standard Project Hurricane shows that this level of protection was known to be inadequate by at least the early 1970s. Since Katrina, elements of the 1950s era plan that were not built , notably the tidal pass closure structures, have been retrieved from

mothballs, and are now being included in virtually all restoration plans. Given this impetus, Team Louisiana used the ADCIRC model to see how these closures would have affected performance during Katrina.

Results do not show the large reductions in surge in Lake Pontchartrain that some have suggested, except along the north shore, where they would certainly have helped. Elsewhere around Lake Pontchartrain, the more important effect would have been to reduce the cumulative volume of flooding through the drainage canal breaches over the next two days. Lake elevation along the south shore with the barriers in place would have dropped to its normal level within hours, instead of taking more than two days.

The lake closure at Seabrook, on the other hand, would have prevented drainage to Lake Pontchartrain of the surge coming in from Lake Borgne and caused more damage due to overtopping and breaching of levees along the IHNC. The model predicts that the greatest increase, over 3 feet, would have been observed just south of the Seabrook structure, but an increase in the surge maximum of a foot or more would be spread over a very large part of the MRGO funnel as water that actually drained to Lake Pontchartrain during Katrina was trapped.

Looking Ahead

The integrated levees and barrier structures now being proposed to provide "Category 5" protection to the GNO by state and federal agencies are similar in many ways to the barrier plan proposed in the late 1950s and authorized after Hurricane Betsy. Clearly, redundant flood protection features can be built to improve reliability. The same can be said about multiple levee lines separated by restored wetland areas designed for short-term storage of surge waters. It appears, however, that most of the proposals being presented continue to rely on legacy levees, which, though they may be built higher, are likely to suffer from legacy flaws. Katrina has taught that the long-term reliability of such structures cannot be assured, given the exposure to surge and wave that must now be assumed.

The experience with the Chalmette levee along the south bank of the MRGO is instructive. The reasons that it failed as early as it did can be debated, whether due to design or construction, but the result is indisputable. Ultimately, the USACE found that it could not retain this earthen structure at the design grade despite two major augmentations in the late 1970s and mid-1980s, and a less extensive rebuilding in the early 1990s. Efforts to improve reliability by armoring the recently rebuilt MRGO levee have been hampered by awareness that the new embankment will require additional lifts at relatively short intervals to counteract erosion and settlement, before it reaches a

less dynamic condition. This remediation would be complicated by the need to repeatedly remove and replace armor installed on the levee slopes and crown.

The only structures that survived on the Chalmette levee run were the water control structures at Bayous Bienvenue and Dupre. They were left by the storm as islands of solidity in a sea of destruction. One of the most important lessons of Katrina was that pile-supported structures like these, as well as T-walls used sparingly elsewhere in the GNO HPS, were capable of surviving the worst that Katrina could deliver.

Team Louisiana members have been excluded from the planning for Category 5 protection now in progress. Some USACE-IPET investigators have apparently been engaged for this work, however, so it is hoped that information derived from study of Katrina levee failures will, one way or another, have an impact on what is built in the future. Our study tells us that failure is not inevitable, but must be actively guarded against. Proposals for more reliably protecting the eastern side of New Orleans can be derived both from study of the GNO HPS failures during Katrina, and from inspection of more reliable structures that have been built elsewhere. Dutch engineers would undoubtedly propose a modular, pile-supported structure like the Oostershelde closure across the 'funnel,' for example, to reduce the threat of surges originating in both of the lakes that flank New Orleans. The Oostershelde closure is an elevated causeway supported by concrete piers providing guides for vertically sliding closure gates that would be lowered only when a storm approaches. It is surely time for this type of creativity, and not just in New Orleans, if we are to honor the 1,500 who lost their lives during Katrina, and avoid more costly mistakes in the future.

Introduction

Louisiana is more vulnerable to hurricanes and major flooding than any other state. Prior to Katrina, 70 percent of the population lived on 36 percent of the land, much of which is at or below sea level. This includes portions of cities such as New Orleans, Houma and Morgan City. It has been known for at least a decade that the bulge of water, or "surge," pushed ashore by a Saffir-Simpson Category 3 hurricane, approaching at a range of speeds from a number of directions, could overtop New Orleans flood defenses on both the east and west banks of the Mississippi (van Heerden 1999, *Times-Picayune* 2002, IEM Inc. 2004, Brouwer 2003).

Hurricane Katrina made landfall in Louisiana as a relatively fast-moving Category 3 hurricane at 6:10 am on Monday, 29 August 2005 (Figure 1).

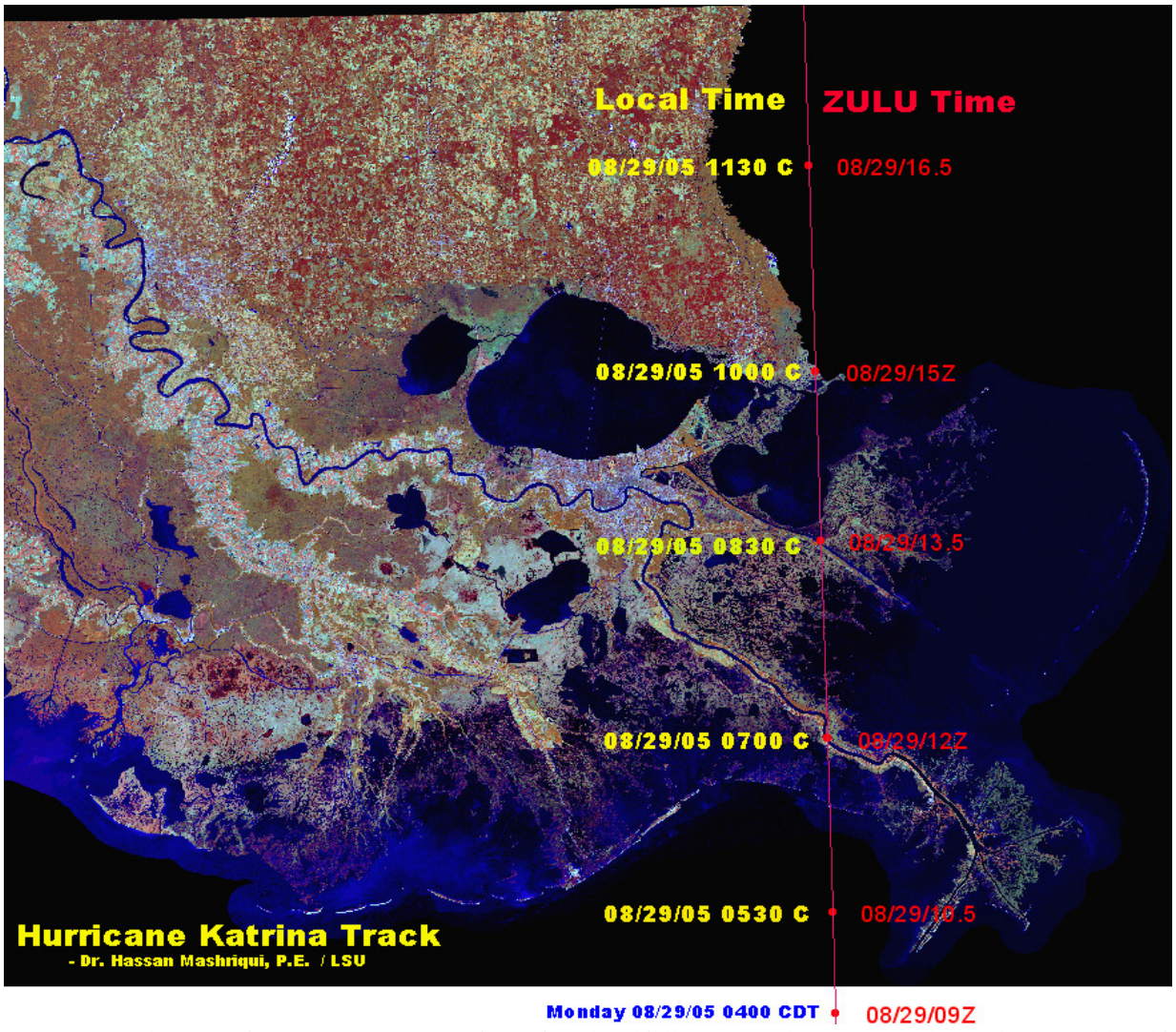


Figure 1. Advance of Hurricane Katrina from first landfall in coastal Louisiana through second landfall in Mississippi (LSU Hurricane Center).

As the storm bore down on Louisiana, LSU Hurricane Center (HC) researchers issued computer generated ADvanced CIRCulation model (ADCIRC) storm surge forecasts, predicting levee overtopping on the east and west sides of the city, to the Louisiana Office of Homeland Security and Emergency Preparedness (LOHSEP) Emergency Operations Center (EOC); a large email list serve; and the media. The Times-Picayune newspaper, in New Orleans, carried a graphic on Sunday morning that showed these forecast results, but also depicted the possibility of overtopping of lakefront levees (Figure 2). It is believed that this information encouraged compliance with evacuation orders.

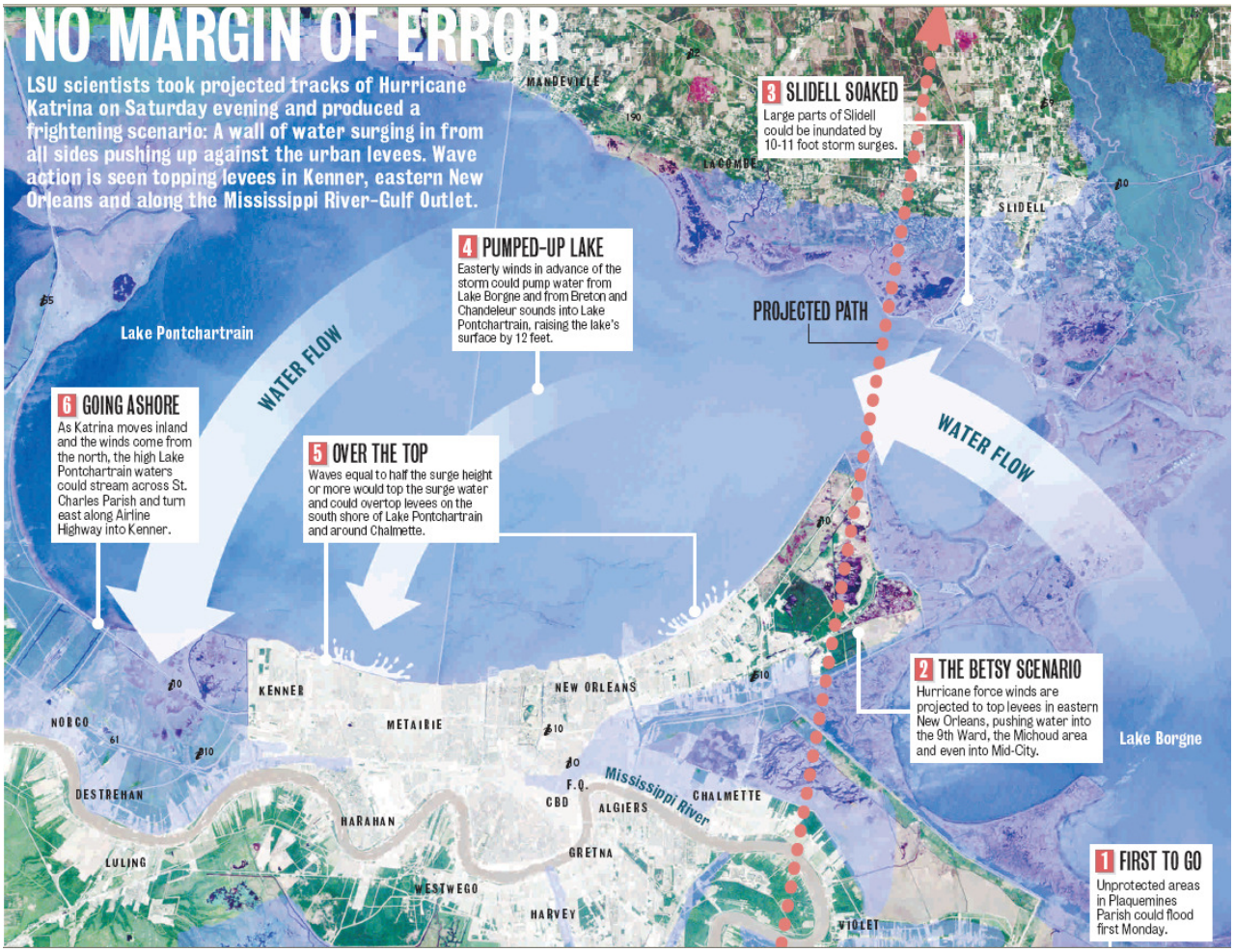


Figure 2. Graphic that appeared in the Sunday morning (August 28, 2005) edition of the New Orleans Times-Picayune newspaper, showing LSU predictions of overtopping (courtesy of the Times-Picayune).

Of the populated areas that constitute Greater New Orleans (GNO), 80 percent of Orleans Parish, 99 percent of St. Bernard Parish, and approximately 40 percent of Jefferson Parish were flooded, in some cases for weeks. This flooding may have either directly or indirectly cost the lives of as many as 1,500 Louisiana residents. However,

the true loss-of-life of this disaster still remains uncertain. The Louisiana Department of Health and Hospitals (DHH) Louisiana Family Assistance Center, which closed down almost a year after the storm, found and reunited thousands but approximately 130 missing remain as of December 2006 (though most are considered drowned); while the New Orleans Fire Department and others continue to recover the remains of deceased victims from the rubble of destroyed homes.

Over 100,000 families were rendered homeless, the great majority of whom had heeded evacuation orders, making this disaster the worst since the 1927 Mississippi River flood in terms of homes destroyed. The hurricane protection system (HPS) that all residents of GNO depended upon for their security from surge floods failed catastrophically with over 50 breachings or breaks. Over 170 miles of the 350 miles of the levees that protected greater New Orleans were either destroyed or damaged (IPET 2006b).

After the storm tore through Mississippi, LSU HC scientists and engineers entered flooded New Orleans accompanied by police teams, to begin trying to reconstruct what had happened. ADCIRC model output guided these teams to specific locations where it was expected that surge conditions would have been most severe. Working initially with just laser levels and available LIDAR topography, they began to compile evidence of storm surge elevations outside the levees and flood walls of the hurricane protection system (HPS), seeking to identify high water marks (HWM) that had been relatively unaffected by wave action. It was quickly determined that the surge model output was remarkably accurate (most hindcast nodes at +/- 15% root mean square error RMSE, nearly 1:1), giving them confidence to begin to rule out some possible causes for key levee and floodwall failures including, at a number of breach sites, the assumption of overtopping-induced erosion. Insights gained in the first weeks after the storm before most other investigators arrived on the scene proved to be critical to understanding what had happened. During this critical period, evidence was observed that was quickly destroyed either in the rush to close breaches or by the natural decomposition of ephemeral features such as debris lines.

Formation of 'Team Louisiana'

The initial goal of the LSU HC field effort was to validate the ADCIRC model forecasts. It quickly became apparent that because of the extent of loss of life and the magnitude of the property damage, any discussion of surge elevations and the related causes of levee failures would not only be controversial, but also a potential basis for legal proceedings. In late September, Mr. Johnny Bradberry, Secretary of the Louisiana Department of Transportation and Development (LDOTD), initiated formation of a Louisiana forensics investigation team under the direction of Dr. Ivor van Heerden,

Director of the LSU Center for Study of the Public Health Impacts of Hurricanes (also known as the Hurricane Public Health Center (HPHC)), to build on the initial LSU Hurricane Center efforts. Secretary Bradberry was joined in these efforts by Louisiana Attorney General Charles Foti.

LSU was commissioned in October 2005 by the LDOTD to assemble a team of Louisiana-based academic and private sector experts to "collect forensic data related to the failure of the levee systems around greater New Orleans" that occurred during passage of Hurricane Katrina on the morning of 29 August 2005. This group, later known as 'Team Louisiana,' was to focus on the hurricane protection system (HPS) designed and constructed over a 40 year period by the U.S. Army Corps of Engineers (USACE) for the East Bank of New Orleans, New Orleans East, and St. Bernard Parish. The LSU HC continued to provide assistance to Team Louisiana through a critical grant from the McKnight Foundation to cover additional team expenses.

By the time Team Louisiana was commissioned, the LSU HC had uncovered apparent discrepancies between their findings and early USACE statements about the causes of levee and floodwall failures (van Heerden & Bryan 2006). Following these reports, at least three other investigations were organized by groups as diverse as the University of California, Berkeley and the American Society of Civil Engineers, as well as by the USACE itself. These external investigative teams included few scientists or engineers from Louisiana. Secretary Johnny Bradberry of the LDOTD saw a need for an official state-sponsored initiative to ensure that state and local perspectives were not ignored as these external investigations proceeded.

An important secondary purpose of Team Louisiana was to energize investigations by the external teams that had greater access to necessary resources, and to encourage these teams to conduct an expeditious and comprehensive review of the causes of the disaster while the evidence of failure was still accessible for study. Team Louisiana members cooperated to varying degrees with the external teams. It is our assessment that the combined, and at times competitive, parallel efforts resulted in an unusually comprehensive treatment of a complex disaster in a relatively short period of time.

Scope of Work

From an engineering perspective, forensics science is the study of materials, products, structures or components that do not operate as intended. In the context of the flooding of New Orleans, the purpose is to understand, first, what performance was expected from the GNO HPS and, second, to identify causes of failure as part of an effort to improve future performance. Team Louisiana was asked, more specifically, to develop a time-history of surge and wave elevations for levee and floodwall

reaches that failed, to compare this information to the designed and actual levee crown and floodwall crest elevations, and to assemble and examine all relevant design memoranda, construction plans and as-built surveys. In addition, Team Louisiana was asked to participate in debriefing of eye-witnesses, assembling stopped clock data, collecting aerial and ground level photographic evidence, and conducting non-destructive testing to determine soil foundation conditions and sheetpile depths in the vicinity of floodwall breaches.

Researchers from Team Louisiana have collaborated to a greater or lesser degree with all of the other investigative teams and have spent many hours in the field together with their members. Team Louisiana took a lead in ensuring that all teams agreed upon and used the same HWM survey data to eliminate any potential for confusion about how high the surge actually got. This was critical because a range of marks were originally available that were contaminated to a greater or lesser degree by wave action. As time progressed and as Hurricane Rita brought a second high water event to the scene on 24 September 2005, it was vital to get this database established before the perishable evidence disappeared. Drs. Stephen Maynord and David Biedenharn of the USACE Engineering Research and Development Center Coastal Hydraulics Laboratory (USACE-CHL) were particularly helpful in ensuring that these efforts were successful. While there has been some confusion regarding mean sea level and geodetic datums, there has been little disagreement among the teams about how high the water got either inside or outside of the HPS.

Composition of 'Team Louisiana'

Dr. Ivor van Heerden, Director of the HPHC, and Deputy Director of the LSU Hurricane Center, was selected to lead the State forensics effort. Dr. van Heerden recruited three other LSU scientists including an oceanographer, a hydraulic engineer and a geotechnical engineer. This group of academic researchers was significantly augmented by the addition of three senior engineers from the private sector, two geotechnical engineers and a water resources engineer, who each had participated in work associated with numerous flood control features in the GNO area across the 40 year evolution of the Lake Pontchartrain and Vicinity Hurricane Protection Project (Table 1).

External Study Teams

The external study teams, particularly the Independent Levee Investigation Team (ILIT) that grew out of the UC Berkeley initiative, and the Interagency Performance Evaluation Team (IPET) sponsored by the USACE, concluded their work and issued final reports earlier this summer (ILIT 2006, IPET 2005, IPET 2006a-c). The USACE also

Table 1. Members of Team Louisiana	
Ivor L. van Heerden Ph.D.	LSU Hurricane Center Department of Civil and Environmental Engineering, LSU
G. Paul Kemp Ph.D.	Natural Systems Modeling Lab, LSU
Hassan Mashriqui Ph.D., P.E.	LSU Hurricane Center Department of Civil and Environmental Engineering, LSU
Radhey Sharma, Ph.D.	Department of Civil and Environmental Engineering, LSU
Billy Prochaska P.E.	Retired professional geotechnical engineer
Lou Capozzoli Ph.D., P.E.	Retired professional geotechnical engineer
Art Theis, P.E.	Retired state construction engineer
Ahmet Binselam, M.S.	LSU Hurricane Center Department of Civil and Environmental Engineering, LSU
Kate Streva, B.S.	LSU Hurricane Center Department of Civil and Environmental Engineering, LSU
Ezra Boyd, M.A.	LSU Hurricane Center Department of Civil and Environmental Engineering, LSU

set up a review board under the auspices of the National Academy of Sciences with the assistance of the American Society of Civil Engineers (ASCE) that has issued findings (ASCE 2005, ASCE 2006a-b).

IPET had virtually unlimited funding and eventually encompassed the work of 150 government, academic and private sector scientists and engineers. The ILIT had very limited funding but grew to include more than 30 of the most capable levee scientists and engineers in the world, most contributing their time and expertise on a *pro bono* basis. The scope of the ILIT work was broader in some respects than that of any other team including ours. They carried out detailed technical investigations of the causes of failures, but also identified possible "dysfunctionalities" in the entire "Technology Delivery System," where they looked at cultural biases and apparent conflicts in and among the various governmental organizations that had responsibilities for designing, constructing and managing the HPS.

The findings of these investigations, and our informal collaboration with the investigators, have been vastly helpful to us. ILIT and IPET results differ in some details, but generally concur on the specific mechanisms of most of the foundation-related floodwall failures. **It is not surprising, however, that from a local perspective, these external probes appeared to miss some of the context in which the design and construction of the still incomplete federal HPS – originally planned to take 13 years at a total cost of less than $90 million – stretched out over 40 years. The project cost nearly $700 million, and consumed nearly $200 million in locally generated funds and services.** Over this time, the USACE provided local sponsors with many conflicting claims, but few reliable assurances of the actual level of protection being provided. **Repairing the GNO HPS to deliver the pre-Katrina level of protection has cost as much in the year since Katrina as was spent in the previous 40 years.** It is not surprising, then, that the relationship evolving over two generations between the USACE and those being protected is complex and easily misunderstood. It is hoped that this report will enrich the historical record and provide additional local perspective.

Team Louisiana Forensics Approach

The surge generated by Hurricane Katrina, at one point a Saffir-Simpson Category 5 storm which eventually came ashore in Louisiana and Mississippi at Category 3 strength, are unprecedented in U.S. history. To give one indication, Katrina generated the largest significant wave height (the average height of the top third of all recorded wave heights) ever measured by a National Data Center Buoy. The recorded wave height was 55 feet, just hours before landfall, registered at a buoy less than a hundred miles south of Dauphin Island, Alabama (Knabb et al. 2006). There is a potential for any forensics investigation to convey an apparent omniscience derived from 20:20 hindsight, and to lose sight of key points like this one. We have tried to avoid this trap by focusing on what was known at the time the GNO HPS was designed, the analytical tools that were available then, and what tools were used.

On the other hand, engineers – then or now -- must all deal with uncertainty and follow accepted practice to account for unknowns that increase the risk of failure. As one of our senior engineers pointed out, it is the anomalous stratum, rather than the average soils condition, that generally causes foundation failure. Engineers address these uncertainties in levee and floodwall design by adding freeboard to raise crown elevation beyond the minimum specified, by inflating the stress to be resisted by a "factor of safety" sufficient to account for unknowns, and by incorporating redundant measures to limit the effect of the failure of a single component. These are some of the key features that distinguish a safe system from one that is unsafe, and are the focus of

this investigation.

Failure can occur when a system is stressed beyond what it was designed to resist. It is important, then, to know whether the design criteria were appropriate given the risks known at the time, and the level of performance required.

It is also important to ascertain whether any particular failure occurred before or after the design limit was attained. It is possible that a stress may occur during an event that challenges the design, but that failure or weakening took place before this critical stress was reached.

Finally, an adequate and appropriate system design may be compromised by poor construction and maintenance. The documentation of fraud or negligence can be critical in these instances, but the path to better future performance is relatively clear.

In another category are failures that take place because a system is under-designed. This situation can arise when risks and failure mechanisms that should have been known at the time are either ignored or underestimated. While an under-designed system may be rendered even less effective by poor construction or maintenance, design deficiencies can ultimately render these factors largely irrelevant. Such a flawed system cannot perform properly regardless of how well it is constructed or maintained. The path to recovery is more difficult when a design problem is discovered than when the issues arise during construction and maintenance. Design failures can occur only within a culture that tolerates them, is incurious about the risks or that does not adequately support its design professionals with training or rigorous management for quality assurance. Correction of these problems may require deep organizational retooling.

Another approach to dealing with uncertainties in engineered systems is to incorporate back-up protection, also referred to as redundancy. Team Louisiana has benefited from previous release of reports documenting findings by ILIT and the USACE IPET. These teams have focused almost exclusively on the federally designed and constructed perimeter HPS, and have described this system as similar to a series circuit in an electrical system, in that the malfunctioning of any component can lead to system failure. The IPET report has, however, described the GNO pumped drainage infrastructure as a potential back-up system that, for a variety of reasons, failed to perform as such.

In fact, the pre-Katrina GNO HPS does include some redundancy in the form of interior levees in East Orleans and St. Bernard that were constructed by state and local

authorities inside the perimeter HPS. These lower levees were built to separate thousands of acres of undeveloped, tidal marsh buffers on the protected side of the federal levees from developed areas under pump (Figure 3). While these levees did not prevent catastrophic flooding during the Katrina event, they did delay the onset of flooding in a way that deserves additional attention as redesign and reconstruction of the GNO HPS proceeds.

Unlike a river flood that may last for months, the challenge presented by a hurricane surge lasts only a matter of hours at any one point. Defense in depth offers a potential to reduce the catastrophic impact of overtopping or breaching at the perimeter.

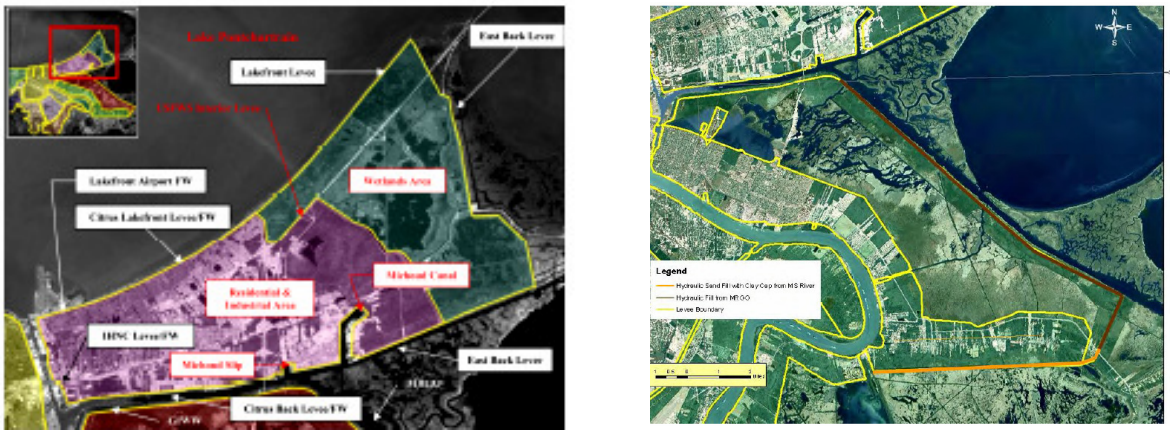


Figure 3. Wetland buffer and storage areas incorporated into the perimeter HPS in East Orleans (above) east of populated area (IPET 2006b, V-18-2) and St. Bernard (below) north of development (IPET 2006b, VII-18-23).

Organization of the Report and Key Questions

This report is organized in two parts. The first part provides background information critical to understanding the physical and historical setting relevant to flood protection and drainage; the magnitude and sequence of stresses placed on the HPS by Hurricane Katrina; and the nature of the flooding that led to the direct or indirect loss of as many as 1,500 Louisiana residents and the destruction of much of New Orleans.

The second part addresses the forensics issues as we see them. The following key questions were formulated to guide the forensics investigation. Each is discussed in further detail in the chapters that follow, with a final chapter providing specific recommendations.

1. Was the GNO HPS properly conceived to accomplish the Congressional mandate to protect against the "most severe combination of meteorological conditions reasonably expected?"

2. Were the levels of protection, or crown elevations, specified in designs for HPS elements sufficient to resist overtopping by surge and waves associated with the 100-year Standard Project Hurricane?
3. Did incorrect design assumptions compromise performance? Should these have been detected and corrected by engineers equipped with the tools available at the time? Were opportunities to install redundant or back-up protection exploited?
4. Did the Mississippi River Gulf Outlet (MRGO), a free-flowing, deep-draft navigation canal that pierced the HPS on the eastern side, compromise system performance?
5. Was the system maintained and operated to assure the required level of protection through time? Specifically, how did the 40-year construction schedule impact system performance?

PART I

SETTING THE STAGE FOR FAILURE

The next three chapters are devoted to (1) a description of the geographic and geologic context within which the GNO HPS was constructed, (2) an explanation of the Hurricane Katrina storm and surge sequence, and (3) a description of the impacts of the storm on specific elements of the HPS and the nature of the flooding that occurred following overtopping and breaching.

CHAPTER ONE

NEW ORLEANS GEOGRAPHY, GEOLOGY AND HURRICANE PROTECTION SYSTEM

Greater New Orleans Setting

New Orleans is a city that straddles the Mississippi River in one of the world's great deltas. All of the world's great commercial rivers have great port cities at their mouths, and so it is not surprising that New Orleans has been one of the oldest continually occupied commercial hubs of the North American continent. Many great ports, including Rotterdam, the world's largest, are also situated in deltas that experience subsidence upon drainage, and have developed significant flood protection systems to allow development below sea level (Figure 4).

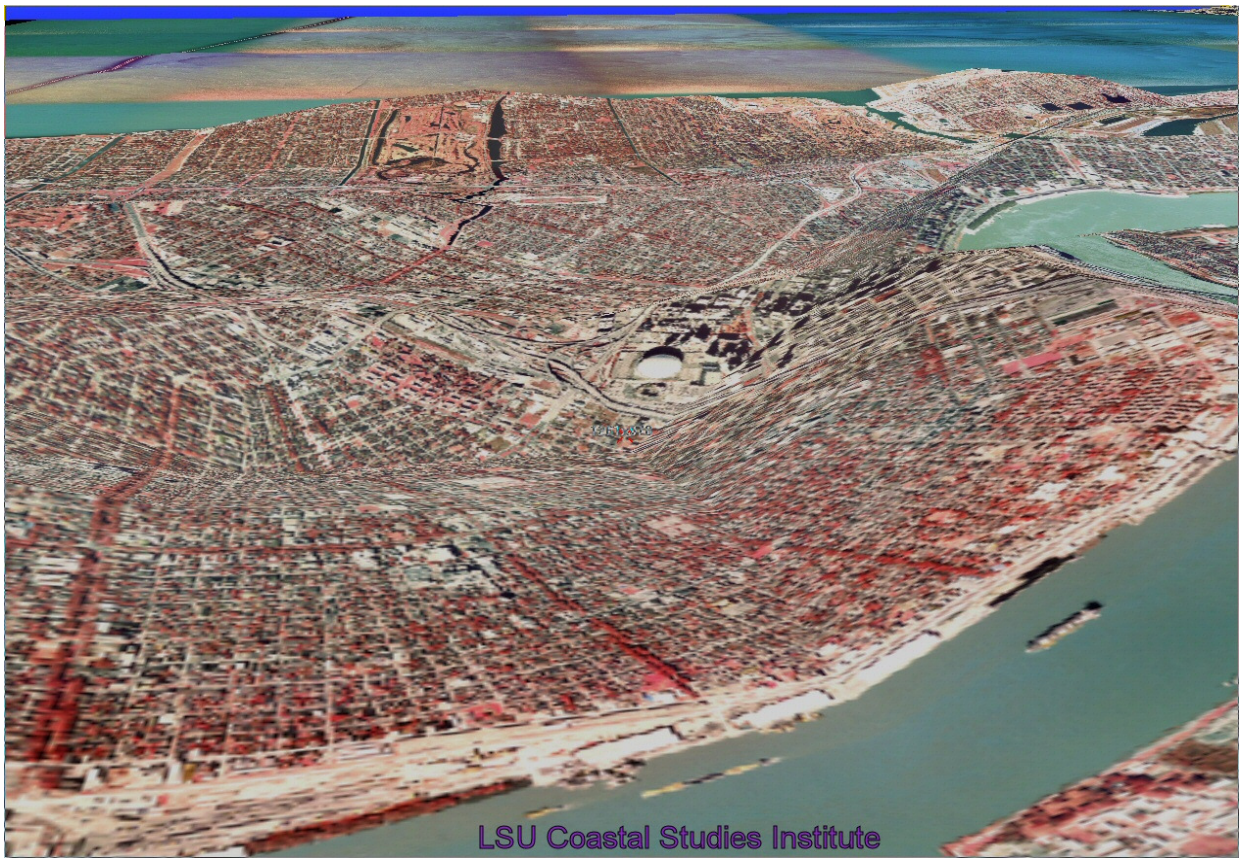


Figure 4. An image of New Orleans draped on LIDAR elevation (with slight exaggeration for effect) by Dewitt Braud, LSU Coastal Studies Institute. Providing a three-dimensional view of the city, higher elevations are shown along the natural levee of the Mississippi in the foreground, the higher lands reclaimed from Lake Pontchartrain in the background, and the Metairie/Gentilly Ridge in the middle. The remainder of the scene comprises drained wetlands that have subsided to below sea level.

While the Mississippi River generally runs from west to east through the metropolitan area, the main body of the city on what is locally called the 'East Bank' lies north of the river. South of the river are the suburbs of the 'West Bank.' Greater New Orleans (GNO) is surrounded by large estuarine embayments. Deteriorating swamps and marshes approach the city closely on all sides but to the north, where it meets the brackish waters of Lake Pontchartrain.

Physical Setting of New Orleans

New Orleans is a city largely below sea level, and completely surrounded by flood-protection levees. Levees reaching more than 25 ft high have been built to protect New Orleans from river floods. Lower levees and floodwalls were constructed in stages to defend from hurricane surges (Figure 5).

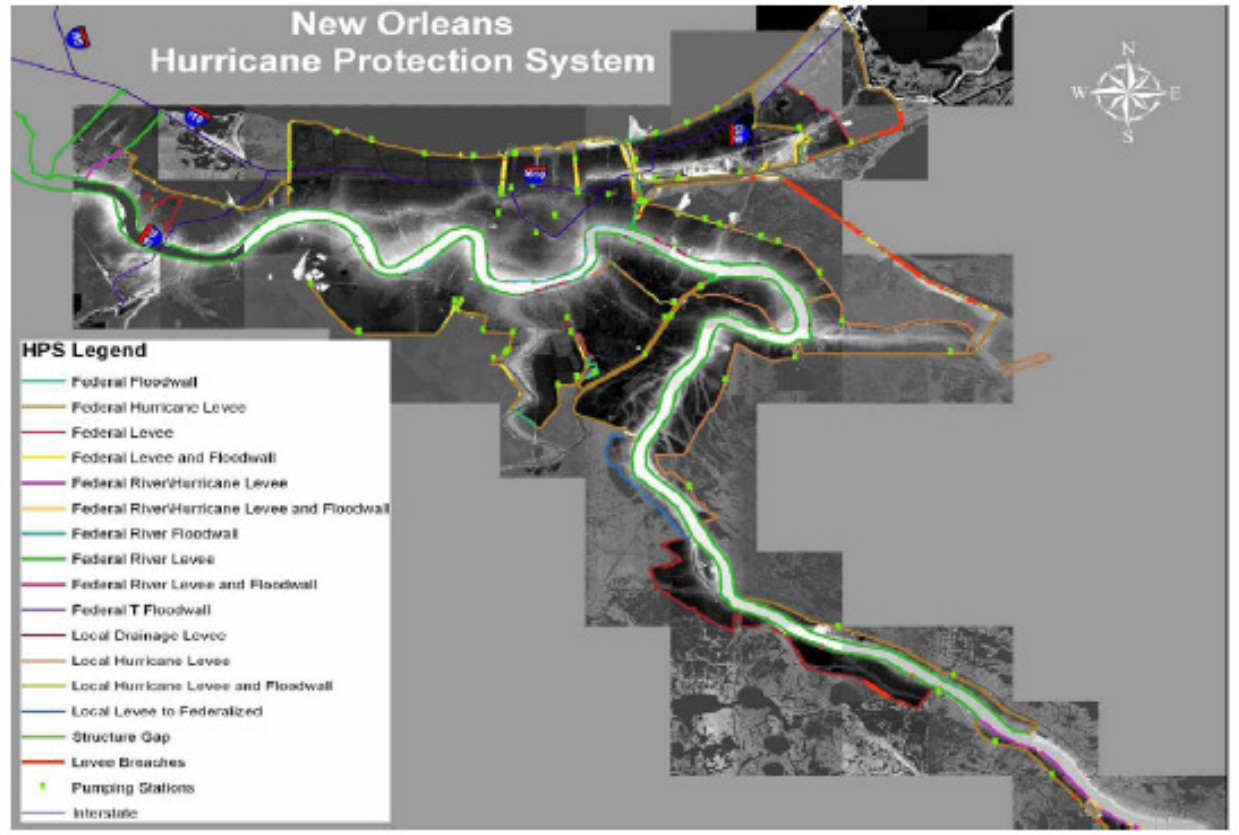


Figure 5. The New Orleans Hurricane Protection System (HPS) showing the East Bank north and east of the Mississippi River, and south of Lake Pontchartrain, the West Bank south of the river, and the Plaquemines system farther downstream (IPET 2006b, Vol. III-12).

The only natural high land, up to 15 ft above mean sea level (MSL), is found adjacent to the Mississippi River, and to a lesser degree on abandoned channels and crevasse deposits that provide the only relief from the low-lying and hummocky bottoms of

reclaimed swamps and marshes (Figure 6). The former wetlands that make up most of the city have responded to drainage by sinking to greater or lesser degrees, but generally from 5 to 10 ft below MSL. Higher areas (5 to 10 ft MSL) along the Lake Pontchartrain shoreline and adjacent to navigation channels like the Inner Harbor Navigation Channel (IHNC), the Gulf Intracoastal Waterway (GIWW) and Mississippi River Gulf Outlet (MRGO) have been formed artificially by placement of dredged sediments beginning in the 1920s.

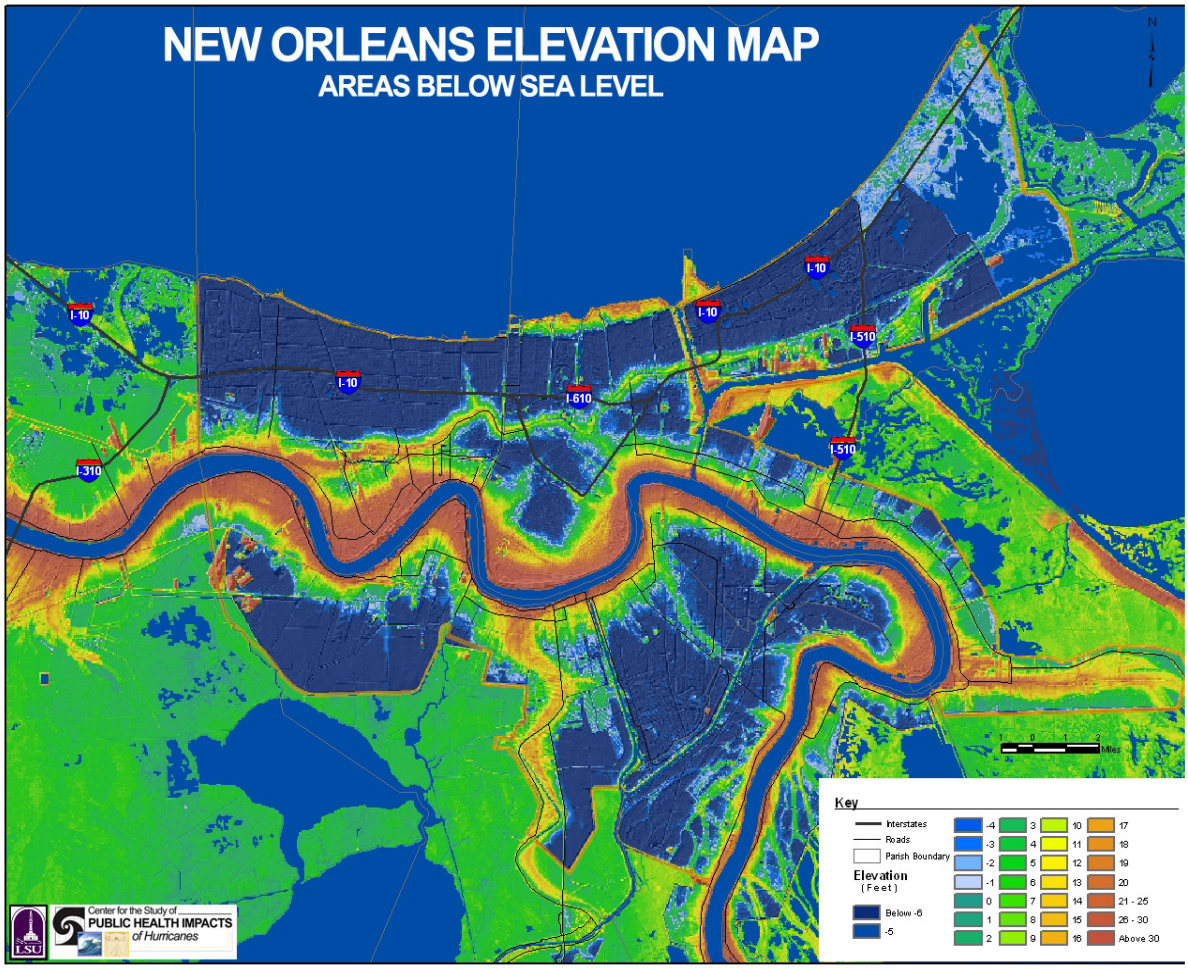


Figure 6. LIDAR mosaic color-coded to show areas above sea level in brown, yellow and green, with areas below sea level as dark blue (HPHC).

Every drop of rain that falls in the city and does not evaporate must be pumped out into Lake Pontchartrain through a series of internal drainage canals connected to pump stations (Figure 7a).

New Orleans East Bank

The GNO East Bank is made up of three parishes (counties) that are governed by separate parish authorities, levee districts and drainage boards. Metropolitan New Orleans (Orleans Metro) includes only the parts of Orleans Parish that lie west of the IHNC, with the exception of the long-settled neighborhood known as the Lower 9th Ward. This part of the 9th Ward was separated from the rest of the city when the IHNC was dredged in the 1920s and is now contained within the same hurricane protection levee that defends suburban St. Bernard Parish.

The federally designed HPS constructed on the East Bank of New Orleans has four large compartments, including suburban Jefferson on the west; the Orleans Metro (that includes the central business district and historic French Quarter); and New Orleans East and St. Bernard to the east of the Inner Harbor Navigation Canal (IHNC) (Figure 7a). The 17th Street Canal, sometimes called the Metairie Relief Canal, separates the area between the River and the Lake into two nearly equal-sized halves (Figure 7a, b). The western half of the east bank was drained and developed later and is governed by Jefferson Parish. A floodwall runs along the western margin of the Jefferson Parish suburbs on the east bank of the Duncan Canal (Figure 7a). This floodwall, composed both of I-Wall and more robust T-Wall segments, faces west toward a large expanse of tidal marsh known as the Labranche Wetlands. The Jefferson compartment flooded partially during Hurricane Katrina but did not experience catastrophic inundation and was cleared of water within a matter of hours once pumping was initiated. IPET reports that flooding in Jefferson occurred as rainwater accumulated in the lowest portions and was augmented slightly (5 %) by water that came through the unattended pump stations from Lake Pontchartrain (IPET 2006c).

In Jefferson Parish, relatively modern pump stations constructed largely with local funds are located at the lakeside ends of the drainage canals that are maintained at water levels well below MSL (Figure 7a). The pump stations are integrated into the perimeter levee protection system**.** Well engineered and substantial concrete levee walls connect the pump stations to the earthen levees along the Lake shore. Interior reaches of the canals in Jefferson Parish have banks that are at or below street level.

The Orleans Metro HPS extends from the IHNC west to the 17th Street Canal, and from the lake to the river. This compartment includes the central business district and Superdome. It includes some of the highest lands along the River (5 to 15 ft), but extensive stretches of very low areas occur just south of the Metairie Ridge and on the north side toward the Lake. This basin averages about 2 ft below sea level. Drainage for much of the heart of the city in the Orleans Metro between the IHNC and the 17th Street Canal is accomplished in a very different way than in the Jefferson Parish

suburbs (Figure 7b). Pumps are located at the innermost city limit of the London Avenue, Orleans and 17th Street Canals rather than at the lakeside perimeter. Accordingly, nearly 20 miles of earthen embankments topped with concrete capped walls have been built down each side of these canals to convey water by gravity from well within the city to the lake. The canals are always open to Lake Pontchartrain, and so these same walls must withstand any surge entering during a storm. A Dutch member of the ASCE *post mortem* team observed the two systems operating side-by-side and asked "Why in the world would you invite the enemy deep inside your own camp?" (van Heerden and Bryan 2006).

The answer to this question is that the Orleans pump stations were in the right place *early in the previous century*, when they were built at the edge of town and at the beginning of an extensive swamp that stretched north to the lake. As the city expanded toward the lake, a feat made possible only as drainage levees and pumps were advanced northward, the stations ended up deeper and deeper "inside" the growing city.

In suburban Jefferson Parish, in contrast, where residential and commercial development has been more recent, it was intended from the beginning that these neighborhoods would extend all the way to the lake. The "legacy" conditions on the Orleans East Bank in some ways set the stage for one the greatest 'natural' disasters to hit the United States. The I-Walls that failed along the 17th Street and London Avenue canals, as well as those along the Orleans Canal that did not, are integral to both the HPS and to the pumped drainage system, as has been described, so that the failures compromised both systems, rendering it impossible after breaching to effectively pump out floodwaters when the storm had passed.

The New Orleans East HPS includes the easternmost suburbs in Orleans Parish located between Lake Pontchartrain and the IHNC (Figure 7c). Orleans East consists almost entirely of drained wetlands and is the "deepest" of the compartments, averaging 5.8 ft below sea level in the area flooded during Katrina. Orleans East includes on its east end an undeveloped formerly tidal wetland covering nearly 22,000 acres that is now largely at or below sea level. This is the Bayou Sauvage National Wildlife Refuge, separated from the developed area by a local levee, and constituting one of the buffer areas that provide some redundancy to the HPS (Figure 6).

Figure 7a. Hurricane protection system and interior drainage system elements of Jefferson Parish on the GNO East Bank. Design level of protection is shown relative to an NGVD29 datum of unknown vintage. Jefferson Parish uses perimeter pumps and interior canals held below sea level. Floodwall along the east side of Jefferson Parish is known as Jefferson Parish St. Charles Parish Return Levee.

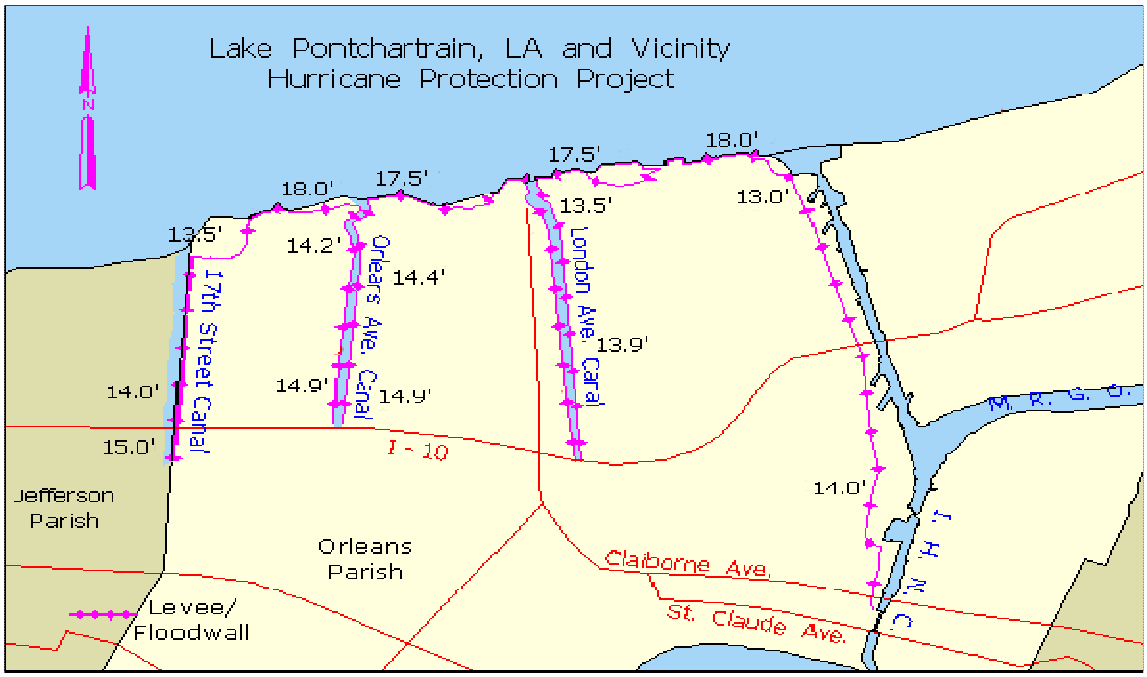


Figure 7b. Hurricane protection system and some interior drainage system elements of Orleans Metro on the GNO East Bank. Design level of protection is shown relative to an NGVD29 datum of unknown vintage. Some Orleans Metro pump stations are located at the south end of lake-level drainage canals while others pump into the IHNC at the perimeter (Source USACOE-NOD).

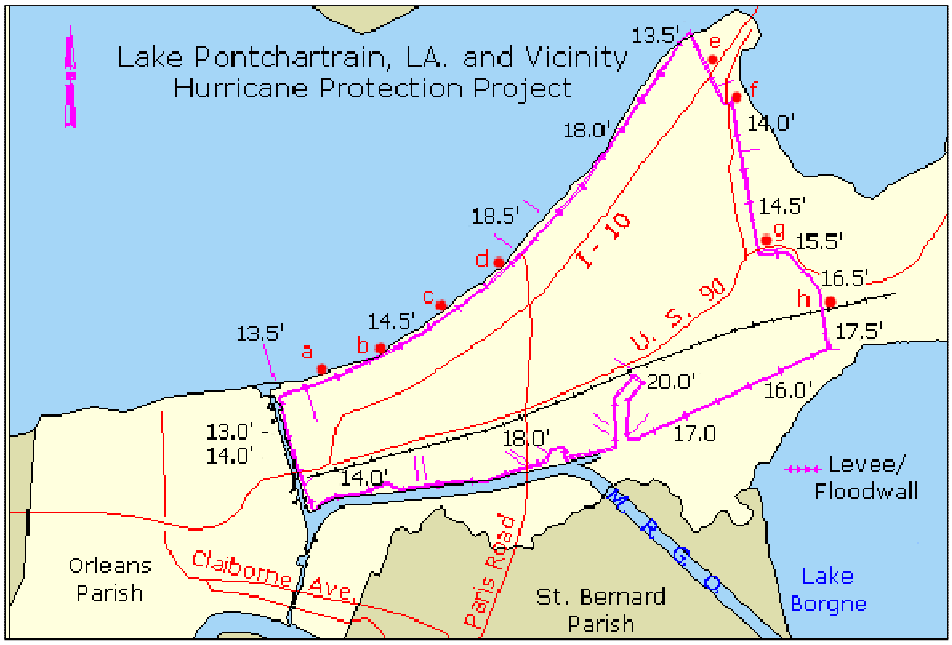


Figure 7c. Hurricane protection system elements of New Orleans East on the GNO East Bank. Design level of protection is shown relative to an NGVD29 datum of unknown vintage. New Orleans East uses perimeter pump stations and interior canals maintained below sea level (Source USACOE-NOD).

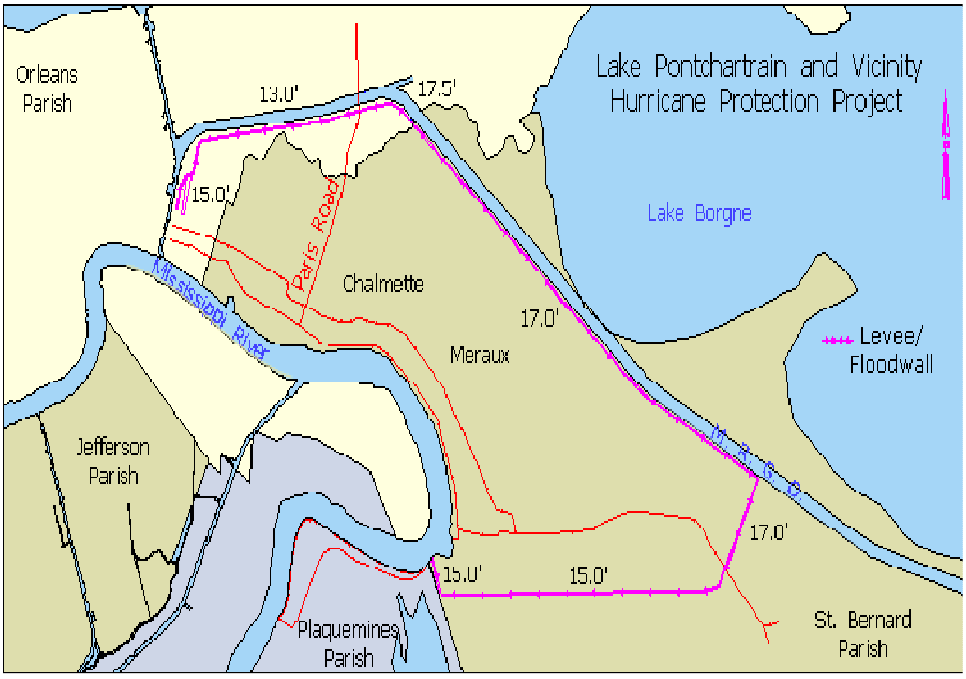


Figure 7d. Hurricane protection system elements of Lower 9th Ward and St. Bernard Parish on the GNO East Bank. Design level of protection is shown relative to an NGVD29 datum of unknown vintage. St. Bernard uses perimeter pump stations and interior canals maintained below sea level (Source USACOE-NOD).

The St. Bernard HPS includes the easternmost areas of the city directly south of Orleans East, between the GIWW and the Mississippi River (Figure 7d). The St. Bernard HPS includes the Lower 9th Ward even though that neighborhood is a part of Orleans Parish, as was mentioned previously. It also contains a large swatch of tidal marshlands, nearly 32,000 acres, that provides a storage buffer located between the main federal hurricane protection levee along the south bank of the MRGO and an inner state-built levee (Figure 6). Two gated structures in the federal levee at the Bayou Bienvenue crossing on the west end, and at Bayou Dupre farther to the east, allow tidal exchange between the wetlands and the MRGO. The developed and drained portion of the St. Bernard polder lies behind a lower state-built embankment, the 40 Arpent Levee, sheltered behind more or less impounded wetlands. Pump stations located along the 40 Arpent levee discharge into the wetlands. The developed portion of St. Bernard that flooded during Katrina averages 2.5 ft above MSL, which is higher than most of the GNO East Bank, following as it does the natural levees of the Mississippi River and some of its abandoned distributaries (Figure 6).

New Orleans West Bank

Three of the GNO East Bank compartments described above flooded catastrophically during Katrina, and remained flooded for as much as 50 days after the storm. The West Bank, which is primarily in Jefferson Parish but includes a portion of Orleans Parish on its eastern margin, is protected by its own HPS, and was fortunate not to be exposed to Katrina's surge. Construction of the West Bank HPS was started later than north of the Mississippi River, is less complete and currently provides a more limited level of protection than on the East Bank. Fortunately, the track of Katrina was such that its winds tended to blow water away from these levees. Hurricane Rita caused more of a flood crisis on the West Bank than Katrina when it made landfall 150 miles to the west on the Louisiana/Texas border, and pushed water levels at the head of the Barataria estuary up by 5 feet, threatening to overtop levees along the Harvey Canal.

Geologic Setting of New Orleans

The geology of New Orleans contributed to the destruction of New Orleans during Katrina, not just because much of it lies below sea level, but because the soils that underlie it imposed significant challenges for engineers designing and constructing the HPS. For this reason, it is as crucial to understand not only what is visible today at the surface but what lies hidden beneath the City.

Mississippi River Delta Evolution

The Mississippi River drains approximately 41 percent of the continental United States, a land area of 1.2 million square miles. The great majority of its bed load is deposited on a well developed flood plain upstream of Baton Rouge. Even so, just about the whole state of Louisiana south of Interstate 10 is new land, a young delta in geological terms. Between 12,000 to 6,000 years ago sea level rose dramatically as the climate changed and became warmer, entering the present interglacial period, which geologists term the Holocene Epoch (last 11,000 years). Sea level rose approximately 350 feet over this time, causing the Gulf of Mexico to advance into southeastern Louisiana and inundate vast tracts of previously exposed coastal lands. By 6,000 years ago sea level had risen to within 10 to 15 feet of its present level.

The modern Mississippi Delta is built on a skeleton of channels, now largely abandoned, that have deposited large quantities of sediment over the banks into near sea level wetlands. Six major depositional lobes, or coalescing zones of deposition, have been identified (Figure 8). Four of these emanate from the Mississippi River itself and two from the Atchafalaya River. In what is now southeastern Louisiana, deltaic sedimentation did not begin until just the last 5,000 years (Saucier 1994) when the coastline was about thirty miles north of where it is today.

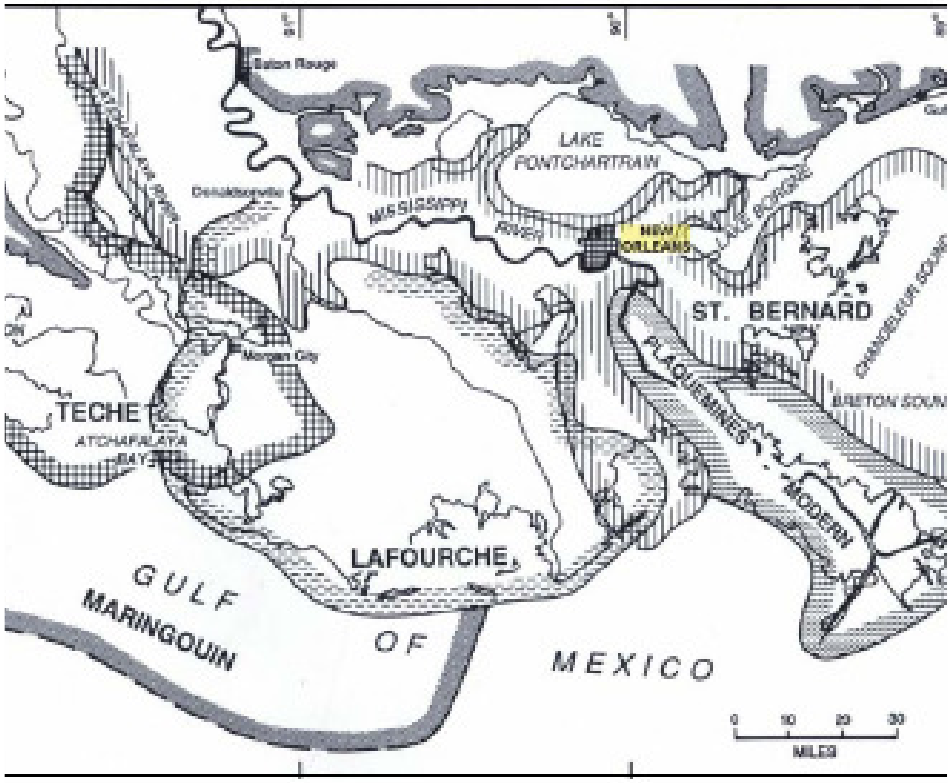


Figure 8. Major Holocene delta lobes of the Mississippi River. Modified from Saucier 1994 and ILIT 2006.

Deltas and rivers, like all natural systems, change over time. A channel extends into the sea until the hydraulic gradient is so reduced that it can no longer efficiently convey sediment out of the mouth (van Heerden 1983). Then, it breaks through the natural levee it has built, through the process of crevassing to initiate another route that provides a shorter and more efficient route to the sea. Part of the older course is then abandoned and a new channel develops. This process of channel switching on the small scale leads to major shifts every 1,000 years, in which the river abandons one whole lobe of the delta and begins a new era of delta building in another location, perhaps as much as 50 or 100 miles away (van Heerden 1994). A delta is built as the river extends its channel into open waters. When floods overflow the channels, sediment accretion builds above sea level to create new land, relatively high along the natural levees adjacent to the channel, and only as high as the tide reaches elsewhere. When the river switches, the previous delta lobe is considered "abandoned," and begins to sink even though some flow may continue down the old distributary channel.

Switching from an inefficient older course to a newer one can take up to 500 years, and even thereafter, the abandoned channel may continue to convey fine-grained clay and silt for many hundreds of years, providing sediment and fresh water to the wetlands initially created when the distributary was the main channel of the river. Two such former courses of the Mississippi, Bayou Lafourche and Bayou La Loutre, were distributaries of the Mississippi River in the late 1600's when Europeans first arrived. The switching from one delta to another has dominated southeastern Louisiana—and, before that, a much larger section of the coast—for millions of years.

Pine Island Barrier/Beach Trend

Sands of a Holocene complex of beaches and barrier islands lie buried 4 to 90 feet beneath the city of New Orleans and extends to the north shore of Lake Pontchartrain. Saucier (1963) named these relic beaches the Pine Island and Miltons Island beach trends. These sands emanated originally from the Pearl River between Louisiana and Mississippi, to the northeast. The Miltons Island beach trend lies beneath the north shore of Lake Pontchartrain while the Pine Island Barrier lies just south of and parallel to the southern shore of Lake Pontchartrain, extending from the western edge of the city nearly 20 miles, beneath the Lakeview and Gentilly neighborhoods of New Orleans, northeast to the Little Woods region of New Orleans East (Figure 9).

The Pine Island Barrier is believed to have been deposited when sea level had almost risen to its present level, about 4,500 years ago. At that juncture, the rate of sea level rise began to slow and an unusually large amount of sand was introduced near the ancient shoreline by the Pearl River. This was spread westerly by waves and currents, in a long linear sand shoal, which soon rose above sea level into a beach ridge along a northeast-

southwest trend (Figure 10) (Saucier 1963; ILIT 2006). The subsequent development of accretion ridges indicates that shoreline retreat halted and the beach prograded southwestward, into what is now the Gentilly and Lakeview area. By about 5,000 years ago, the beach had risen sufficiently to form a true barrier spit anchored to the mainland near the present Rigolets, with a large lagoon that later became Lake Pontchartrain enclosed on its northern side. Sometime after this spit formed, distributaries of the Mississippi River began depositing deltaic sediments seaward of the beach trend, isolating it from the Gulf of Mexico. The Pine Island Barrier was subsequently surrounded and buried by sediment. The base of the Pine Island sands have subsided 25 to 45 feet over the past 5,000 years.

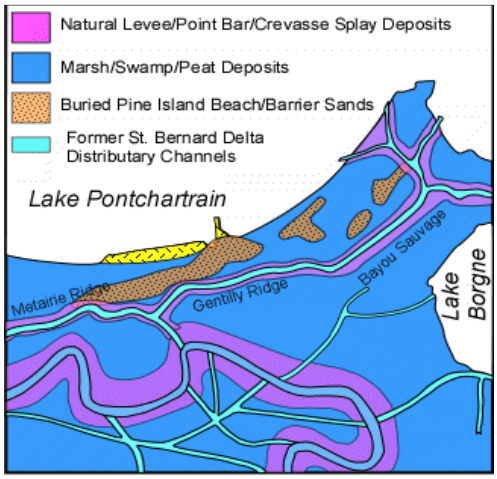


Figure 9. Pine Island Barrier, New Orleans (from S. Nelson, Tulane University http://www.tulane.edu/~sanelson/Katrina).

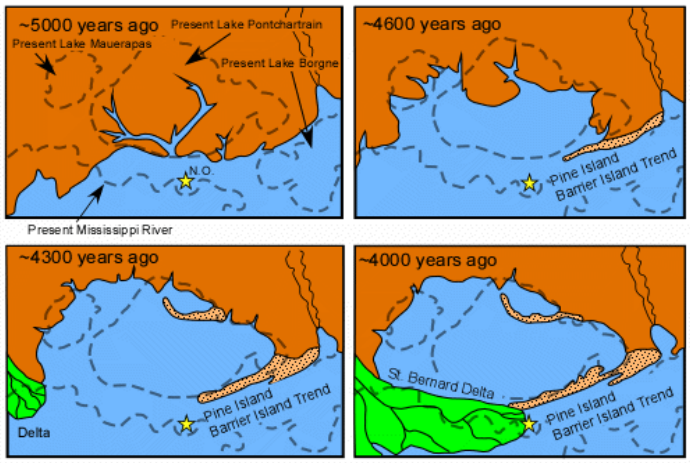


Figure 10. Evolution of the Milton and Pine Island barriers (from S. Nelson, Tulane University http://www.tulane.edu/~sanelson/Katrina).

The Pine Island sands reach thicknesses of more than 40 feet in the Gentilly area, but diminish to the west towards the Lakeview area, pinching out close to the 17th Street Canal breach at the New Orleans/Jefferson Parish boundary (Figure 11). They were present on portions of the bed of the London Avenue Canal and, because they were porous, provided an avenue for water to seep under the canal floodwalls when Katrina raised the water level in the canal.

These are the sands that were eroded and deposited amongst the houses at the two London Avenue breaches during Hurricane Katrina (Figure 12). The Pine Island beach sands created a natural border that helped form the southern shoreline of Lake Pontchartrain, along with deposition by the Mississippi River into the area sheltered behind the ridge. Lake Pontchartrain was still a partially enclosed bay until about 3,000 years ago (Figure 10) when the opening was reduced to narrow tidal passes by the

elongation of the St. Bernard delta lobe (Kolb, Smith, and Silva, 1975).

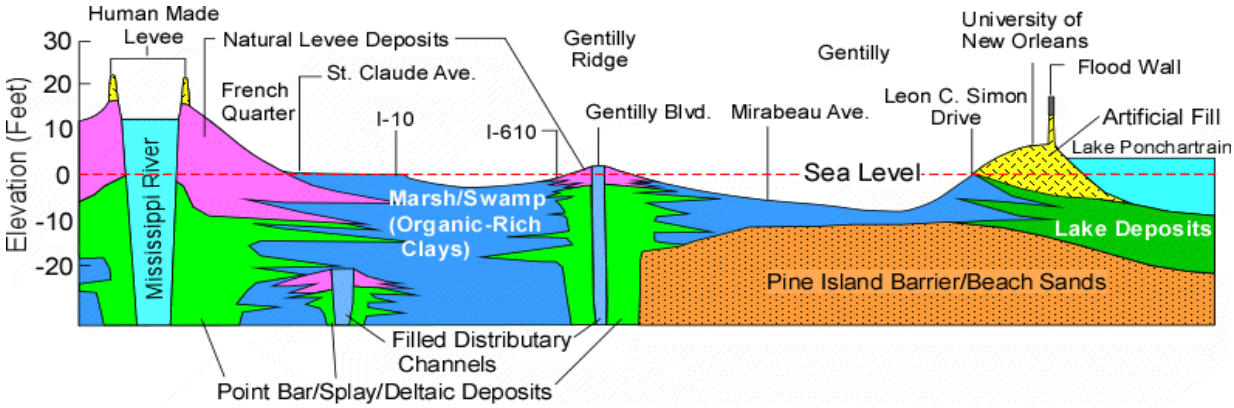


Figure 11. cross section of New Orleans extending from the Mississippi River adjacent to the French Quarter to Lake Pontchartrain near the University of New Orleans campus (from S Nelson, Tulane University http://www.tulane.edu/~sanelson/Katrina)

Figure 12. Photo of clean barrier island sands in dunes up to 6 feet thick near the Mirabeau Breach (east) on the London Avenue Canal.

Interdistributary Zones

Most of New Orleans' residential areas lie within what are called interdistributary basins formed between the deltaic channels. The homes are built on clay and organic rich soils that were once lacustrine (lake), swamp, and marsh deposits. Interdistributary sediments are deposited in low lying areas between the channels and old deltas of the Mississippi River. Sediment charged water spilling over natural channel levees tends to drop its coarse sediment closest to the channel (e.g. Metairie and Gentilly Ridges) while the finest sediment settles out in shallow basins between the distributaries. Fine-grained sediment was also carried into the interdistributary basins when river floods caused breaks in the natural and later man-made levees, well upstream. Sediments were also introduced into these low swampy areas by hurricanes.

Considerable thickness of interdistributary clays can be deposited as the delta builds seaward. Kolb and Van Lopik (1958) noted that interdistributary clays often grade downward into marine clays and upward into richly organic clays of swamp or marsh deposits. True swamp or marsh deposits only initiate when the water depth shallows sufficiently to support vegetation (e.g. cypress swamp or grassy marsh). The interdistributary zone is typified by organic clays, with about 60 percent by volume being inorganic or "fat" clays, and 10 percent or less being silt, usually in thin layers called lenses or stringers by geologists and foundation engineers (ILIT 2006). Kolb and Van Lopik (1958) reported cohesive strengths of interdistributary clays as ordinarily ranging between 100 and 400 pounds per square foot (psf), depending on how deeply they have been buried and compressed. Relatively recently deposited interdistributary clays, like those generally found in New Orleans, can also contain weaker layers in which water has been trapped. Such clays are said to be underconsolidated because the presence of trapped water does not allow them to gain strength as they are buried. These are the anomalous layers that the prudent geotechnical engineer looks for, and designs for.

Engineering Challenges of New Orleans Geology

Ninety percent of the GNO is covered by highly organic and unconsolidated swamp or marsh deposits that are characterized from an engineering standpoint as poorly suited to support the loads imposed by adding buildings or levees or other structures to the surface. As a consequence, virtually every building built in New Orleans today is supported by pilings or piles that are driven into the ground as much as 150 feet. The average home in many areas may be supported by piles 50 to 80 feet long. So, the geology of New Orleans dictates that foundations, piers, docks, levees, floodwalls, indeed almost any structure, require detailed studies of the underlying soils by qualified geotechnical engineers as a prerequisite to construction. Only then can a suitable foundation be designed that will not settle or fail when subjected to normal

loading and use. For the levees and floodwalls of the GNO HPS, the critical loading that guides the design occurs only under that rare situation when a hurricane pushes an extra 10 or 15 ft of water against such a structure. Under these circumstances, the foundation must do more than merely support the levee, it must also ensure, first, that water does not move under the structure through porous layers, and, second, that the supporting strengths of water saturated layers are not weakened by a sudden buildup of pressure within the pore spaces.

The science of geotechnical engineering and foundation design has been challenged to deal with the geology of New Orleans since the founding of the city nearly 300 years ago. Prudent engineers have developed conservative approaches, in some cases after learning from failures, which have allowed successful construction of the Mississippi River levees and buildings tens of stories high. All practitioners agree that it is possible to build levee and floodwall foundations that will not fail when subjected to hurricane surges. When foundation failures occur under critical levees and floodwalls and the surge level is below the crown, as happened in at least four locations during Katrina, it must be presumed that mistakes were made. The same criticism should not be lodged against the foundation engineer when a levee or floodwall is overtopped, however, either because the crown is too low or the surge is too high. This is an important distinction because it has not been customary engineering practice in New Orleans or anywhere in the United States to design flood protection structures to survive overtopping.

CHAPTER TWO

HURRICANE KATRINA AND HER SURGE

Katrina Meteorology

Hurricane Katrina reached maximum winds of 170 mph (1 minute average at 33 ft above ground) at 1300 CDT (1:00 PM) on Sunday, 28 August 2005, about 200 miles southeast of the mouth of the Mississippi River (Knabb et al. 2006). This was Category 5 on the Saffir-Simpson Scale (Appendix 1). The wind field expanded throughout the day until hurricane-force winds extended over 100 miles from the center, so that at one stage Katrina was a very large Category 5 storm. Katrina did not turn northward until the first hours of Monday morning, 29 August, but weakened quickly as it moved toward the coast. The reasons for this weakening are not well understood, but it is typical of intense northern Gulf coast storms. The hurricane made landfall at 0610 as a Category 3 storm with estimated maximum winds of 117 mph (Howard pers. comm.) near Buras, Louisiana.

At the time of landfall, Katrina was reported to be a Category 4 storm. It was not downgraded to Category 3 until the National Hurricane Center (NHC) had completed a careful analysis of all available data and issued its Tropical Cyclone Report in December (Knabb et al. 2005). Knabb et al. (2005) note that the maximum wind at landfall was lower than expected for a hurricane with a central pressure as low as 920 mb. Hurricane Rita struck western Louisiana less than one month later (24 September 2005) and reached an even lower central pressure before it made landfall. Although Katrina's central pressure remained low, surface wind speeds at landfall were below the values that had been predicted from flight level readings.

Track, Winds and Central Pressure

Katrina followed a track due north that took it across the Mississippi River to a final landfall near the mouth of the Pearl River at the Louisiana/Mississippi border at about 0945, 3.5 hours after the Buras landfall. The central pressure rose gradually as the storm was traversing southeast Louisiana. At the time of the Mississippi landfall, pressure had reached 928 mb and Katrina was still a Category 3 hurricane with estimated maximum winds of 105 mph (van Heerden and Bryan 2006, pg 70). The hurricane accelerated after its Louisiana landfall from about 13 to 16 mph, making it a relatively fast-moving storm.

Katrina weakened rapidly after moving inland over southern and central Mississippi, becoming a Category 1 hurricane by 1300, about 3 hours after the

Mississippi landfall. The NHC downgraded Hurricane Katrina to a tropical storm about six hours later, when the remnants of the eye were northwest of Meridian, Mississippi.

The center of Katrina's eye, which was estimated at 25 to 35 miles in diameter, passed about 40 miles east of downtown New Orleans, but the city did not experience winds stronger than Category 1 or weak Category 2 (95 mph) velocities. Katrina's strongest winds were in the right-hand leading quadrant that was almost continually over water. A few instrumented towers placed in various locations in the GNO metropolitan area by the Florida Coastal Monitoring Program (FCMP) and by Texas Tech University measured sustained winds in the range of 71-79 mph (Knabb et al. 2005). A National Weather Service (NWS) anemometer located at an elevation of 52 ft along the Lake Pontchartrain Causeway 9.3 miles north of New Orleans measured a 1-minute average wind speed of 79 mph.

Rainfall

Rainfall along the northern Gulf coast was most intense along and just west of the track, averaging 8-10 inches of rain across southeastern Louisiana and southwestern Mississippi. The National Climatic Data Center reports that the Slidell NWS office measured nearly 12 inches, and that five stations from the GNO measurements ranged from 6 to 13.9 inches (Graumann et al. 2005), suggesting an average of about 9 inches over the 24 hours that included passage of Katrina. This is sufficient to rank the Katrina rainfall among the top rain events reported from New Orleans, above TS Isidore in 2002 (4.5-7.5 inches) and below TS Allison in 2001 (14.5-21.5 inches). Such precipitation totals would have produced street flooding in New Orleans like these earlier rainfall events if the breaches had not turned inconvenience into disaster. Louisiana appears to have been spared tornadoes, in contrast to Mississippi where 11 were reported.

IPET estimated 24-hour rainfall for the GNO, primarily using radar data, between 9.9 to 13.6 inches, beginning at 2300 CDT on 28 August and continuing for 24 hours. They report the rate of precipitation rising to a peak of almost 2 in/hr between 0300 and 0400 29 August, then dropping to 0.6 in/hr for the two hours immediately before the Louisiana landfall in Buras (0600) based on radar estimates. Rainfall is believed to have intensified again following this lull to 1.5 in/hr for three hours (0900). This was the end of the most intense rainfall period as the estimated rate dropped to about 0.5 in/hr for the next 5 hrs (1300) and decreased again to less than 0.2 in/hr after that.

This radar-derived chronology suggests that there were lulls around the heaviest

rainfall periods between 0400 and 0600 or just before dawn, and after 0900 when eyewitnesses would have emerged from their homes to assess the situation. Relatively few eyewitnesses reported from the daybreak interval, unless they were being flooded at that time as in the Lower 9th Ward, but their reports were more frequent after 0900.

Meteorological Assessment of Storm Surge and Waves

If the maximum wind speeds in the GNO area were less than expected, the magnitude Katrina's storm surge was large. Katrina pushed ashore a surge of nearly 30 feet in the vicinity of Waveland and Pass Christian, Mississippi, and reached almost 20 feet against the Mississippi River levees in Plaquemines Parish. The highest surge observed in the GNO was against the MRGO levee in the funnel formed by the convergence of the levee systems built along the north bank of the GIWW and the south bank of the MRGO. The surge penetrated six miles into the GNO through the portion of the GIWW that was enlarged as part of the MRGO project.

The surge also entered the Lake Pontchartrain basin, raising Lake Maurepas by more than three feet 40 miles from the coast. It appears to have penetrated at least six miles inland in many portions of coastal Mississippi as well, and up to 12 miles inland along bays and rivers, crossing Interstate 10 in some locations. Katrina produced a significant storm surge along the eastern Gulf coast into the Florida panhandle. Hurricane-induced flooding extended several miles inland from the Gulf coast in Mobile Bay where the surge was about 10 feet, only about 2 feet lower than it was in Lake Pontchartrain.

Although the storm surge was higher to the east of the path of the eye of Katrina, water level also rose in the Pearl River marshes and in Lake Pontchartrain. The level of Lake Pontchartrain had been rising for about a day prior to landfall in response to the strong easterlies that preceded the storm, reaching about 3 feet (NAVD88). Then the elevated water in the Lake was sequentially tilted first toward the south shore by strong north to northwest winds around 0900, and then toward the northeast quadrant of the Lake (Slidell area) by the trailing westerlies (1200) of the cyclone.

The disastrous effect of the first Lake surge on the I-walls along the GNO drainage canals is well known. The second surge severely affected communities along the northeastern shore of Lake Pontchartrain from Mandeville to Slidell, but is less well understood. It is not known, for example, how much of the surge observed was an isolated Lake Pontchartrain event, and how much was a spillover from the more massive Mississippi surge to the east.

Knabb et al. (2005) explain the magnitude and extent of the storm surge produced by Katrina primarily as a function of the huge size of the storm, even though it had weakened from Category 5 intensity the previous day to Category 3 at landfall. Katrina had a 35 mile radius to maximum winds and a very wide swath of hurricane force winds that extended at least 87 miles east from the center on the day it made landfall. Even though Hurricane Camille (1969) was more intense than Katrina at landfall and followed a similar track, Camille was far smaller. Though Camille produced a similar storm surge maximum just to east of the eye, it did so over a much shorter stretch of coast. It is also probable that the loss of Louisiana coastal wetlands and degradation of barrier islands along Katrina's track in the 35 years since Camille exacerbated the surge in 2005. Additionally, the man-made Mississippi Rivers levees, as will be discussed shortly, exacerbated the surge.

While it was a Category 4 to 5 hurricane offshore, Katrina built and set in motion an energetic wave field. NOAA National Data Buoy 42040 located about 74 miles south Dauphin Island, Alabama, reported a significant wave, the average of the highest one-third of waves measured, of 30 feet as early as 1900 on August 28, 11 hours before the Louisiana landfall, and later measured a record 55 feet at the time of landfall.

Among the most important contributions of the IPET team was a very comprehensive modeling analysis of the waves that reached the GNO HPS (IPET 2006b). IPET found that the height of the waves affecting the south shore of Lake Pontchartrain and reaching the east side of the GNO HPS from Lake Borgne were limited by frictional drag in shallow water (depth-limited) and were not greater than anticipated in the original oceanographic analysis based on the "standard project hurricane" (SPH). Waves in Lake Pontchartrain were relatively high frequency (period less than 7 second) as would be expected for waves generated in the lake itself.

On the other hand, the IPET wave model predicted that a significant portion of the waves that affected levees and floodwalls bordering the funnel east of the GNO had periods twice as long, up to 15 seconds, and must have been generated well offshore. Such waves have the potential to cause significantly greater run-up on levees than the shorter-period (5-7 sec.), locally-generated waves of the same height that were anticipated in the original design. For waves breaking on a beach or a levee, run-up is a function of wavelength, and wavelength increases as the square of the period. Longer period waves generate higher run-up basically by pushing a greater volume of water against the levee in each wave cycle.

Storm Surge Height

By the time Slidell flooded, Katrina had already breached levees in the GNO that ultimately inundated more than 80 percent of the East Bank. The USACE did not entirely succeed in removing these floodwaters until October 11, 43 days after Katrina's landfall, after a second reflooding through the IHNC of the Upper and Lower 9th Ward and Gentilly by Hurricane Rita (24 September). Sheet pile walls driven across the beds of the Orleans Metro drainage canals kept those partially repaired breaches from being reactivated by Rita, but breaches on the IHNC flowed again.

The Hurricane Public Health Center at LSU has worked with Joannes Westerink at the University of Notre Dame since 2002 to develop an operational version of the ADvanced CIRCulation (ADCIRC) storm surge model for the Louisiana coast (van Heerden et al. 2002, 2003, and 2004). ADCIRC, like the SLOSH model used by the National Weather Service (to be discussed in more detail in the next chapter), can predict surge elevations resulting from hurricanes, and is used to uncover details about the oceanographic response to a storm with a given track, speed and intensity. Surge is often discussed in association with hurricanes but is often misunderstood.

Surge is the rise of water above the normal tide level (still water level) generated by the prolonged and changing stresses of hurricane winds on the water surface. Oceanographic and meteorological factors other than the wind stress contribute to surge, like the rise of the ocean under an area of low atmospheric pressure (inverse barometer effect), and an increment due to the momentum transfer toward the coast caused by waves, but wind stress is the most important.

The combined effect of these factors vary greatly from place to place through time and are highly influenced by the geometry of the receiving coast and the man-made structures like levees and floodwalls that are put in place to contain the surge. Only sophisticated models like SLOSH and ADCIRC are capable of simulating these complex interactions over the very large expanses of ocean traversed by a storm like Katrina, while including sufficient detail in the coastal areas of most interest (Figure 13).

Modeling Surge with ADCIRC

ADCIRC is a two-dimensional, depth-averaged, finite-element hydrodynamic model that is forced by a Planetary Boundary Layer (PBL) hurricane wind model (Westerink et al. 2004). The model predicts water surface elevation and currents generated by wind and pressure. LSU forecasting ADCIRC version S08 consists of over 300,000

computational nodes in a domain that includes the western half of the Atlantic Ocean, the Caribbean Sea and the Gulf of Mexico (Figure 13). More than 80 percent of these nodes are concentrated within Louisiana's wetlands and estuaries to reproduce the complex geometry of natural and man made channels and levees that affect surge transmission, amplification and attenuation. The goal was to create a model that could be run quickly and reliably for real-time operations support.

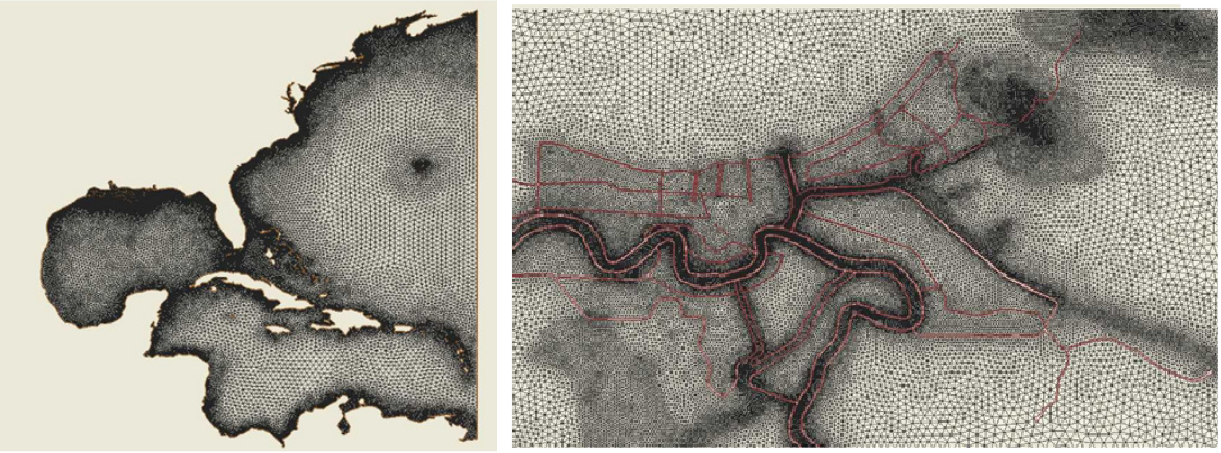


Figure 13. ADCIRC "S08" computational model domain (left) and detailed mesh outlines (right) around New Orleans area (for S08 grid reference see Feyen et al. 2002).

The GNO HPS was represented in an idealized way in that the levee and floodwall heights were presumed to be at the design levels relative to the NGVD29 vertical datum historically used by the USACE New Orleans District. The model was developed with funding from the USACE New Orleans District and used the information they provided, however inaccurate with respect to deficiencies. It is now known that most levee and floodwall crowns were lower than the design level of protection, as will be discussed in Chapter 5. This version of ADCIRC is helpful from a forensics standpoint in providing an estimate of the more limited flooding that would have occurred during Katrina, if the East Bank HPS was complete and more or less at the 1965 authorized level of protection. In fact, it has required more than $500 million in repairs, raising and rehabilitation since the storm to bring limited lengths of the HPS nominally up to the authorized condition.

We have used ADCIRC for forensics purposes to develop a way of stitching together the pieces of the Katrina story provided by eyewitnesses, gages and HWMs, and as a way of separating Katrina impacts that the HPS should have defended against, from those that it could not be expected to stop. More specifically, we wanted to separate flooding over the crowns of levees and floodwalls caused by surge and waves in excess of the design level of protection, from flows that occurred because either the crown

elevations were below the design elevation, or flood protection had been compromised by breaching.

A good surge prediction requires very good information on hurricane track, speed of advance and intensity. The final NHC track has the eye center passing 6 miles to the east of the easternmost St. Bernard levee defenses, so that only the western, weaker eyewall winds affected any populated portion of GNO (Figure 14). Katrina dealt GNO a glancing blow when it veered across the Mississippi River deltaic plain as a relatively fast-moving Category 3 storm. Much more damaging Category 3 storm scenarios are possible that would have flooded the West Bank as well as the East Bank. (www.hurricane.lsu.edu)

A hindcast ADCIRC surge prediction was prepared (Figure 14). Based on this prediction a surge would have entered the GNO HPS before 0800 through the artificial funnel formed by the convergence of levees built along the north bank of GIWW and the south bank of the MRGO (Figure 14). This surge would then have sequentially overtopped levees along the western reach of the combined MRGO/GIWW, and entered the IHNC as a high-velocity jet (Figure 15). Minor overtopping was predicted along the southern St. Bernard levee (Caernarvon) and none on the Lake Pontchartrain shoreline. ADCIRC predicted a potential for overtopping along the Duncan Canal floodwall protecting Jefferson on the west side.

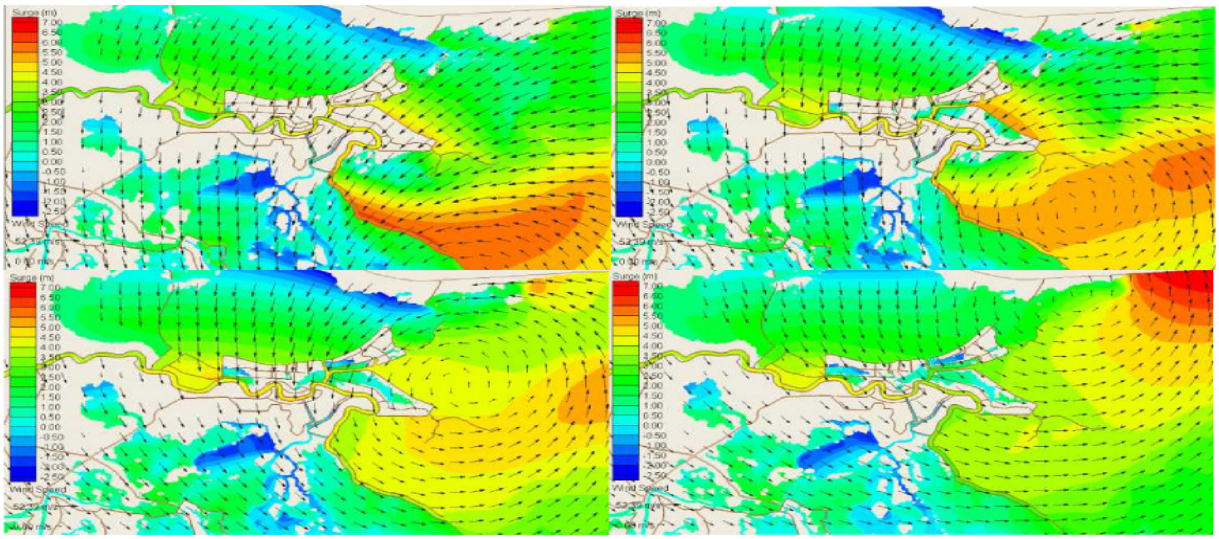


Figure 14. ADCIRC hindcast of surge sequence from August 29 0700 (top left), 0800 (top right), 0900 (bottom left), 1000 (bottom right). Surge is in meters. The vectors indicate wind direction with the center of circulation (eye) visible in the top right and bottom left panels. Overtopping begins before 0800 (HPHC 2005).

Along, the MRGO on the south side of the funnel, water is shown spilling into a large impounded marsh (St. Bernard Wetlands) that is bounded on the south by a secondary state levee. This secondary levee (40 Arpent levee) is from 7 to 10 ft high, compared to the 17.5 ft federal levee along the MRGO, and is primarily used to facilitate drainage of the populated portion of St. Bernard. This secondary levee is depicted as submerged in the vicinity of the Lower 9 th Ward, a portion of Orleans Parish that falls within the St. Bernard drainage basin. The only overtopping shown in the hindcast that did not take place on the morning of August 29 is the water shown entering Jefferson from the western marshes.

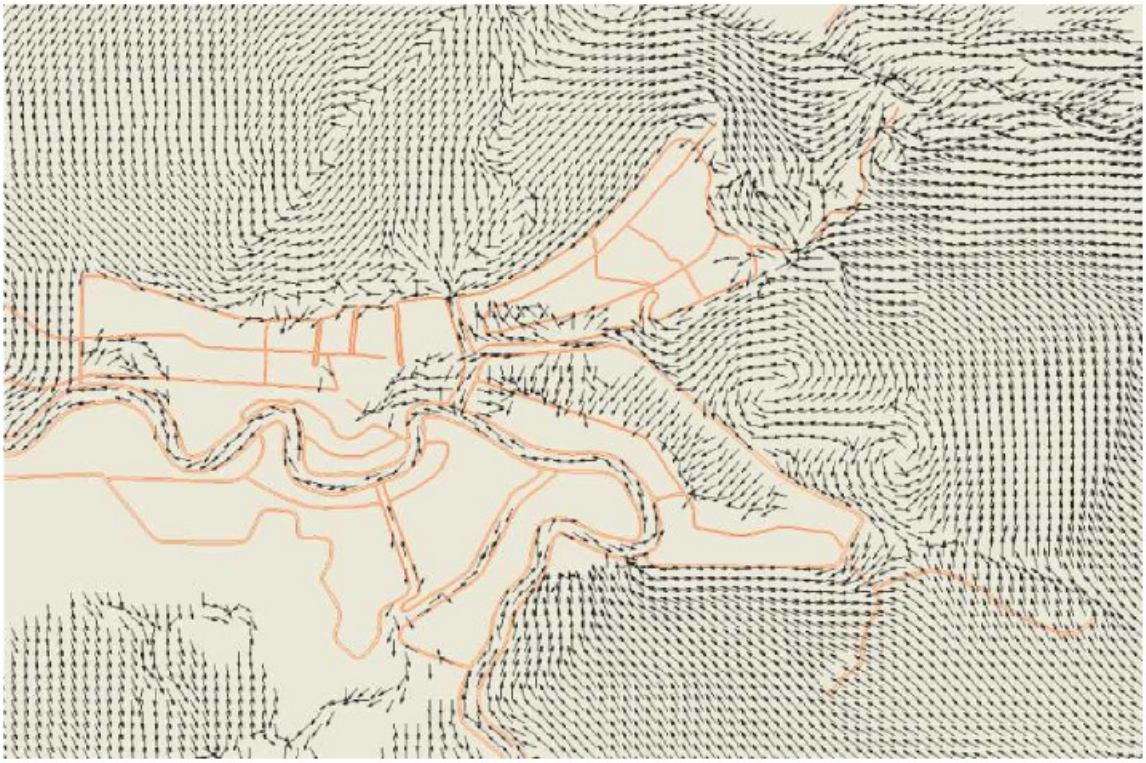


Figure 15. Flows generated by Katrina toward and into New Orleans over the tops of levees near the onset of flooding. Note that circulation in the triangular "funnel" that opens to the east of the IHNC includes flows both normal and parallel to the levees and floodwalls built along the south bank of the MRGO and north bank of the GIWW (LSU HC 2005).

Comparison of Model Results to Observed High Water Marks

Only one complete contemporaneous record of stage exists from the GNO for Katrina (Figure 16). This was a set of visually collected hourly readings recorded by the lockmaster at the southern terminus of the IHNC (IPET 2006b). No automatic recording gages around the GNO functioned through the entire event to document the surge peak. The peak surge recorded at the IHNC lock was 15 ft NAVD88, but the elevation was later reduced to 14.2 ft when it was found that the gage was set 0.7 ft high (IPET 2006b, Vol.

IV-33). But the local mean sea level (LMSL) elevation is about 0.7 ft above the NAVD88(2004.65) datum plane, so it appears that the surge got to 15 ft above the local sea level, essentially what the 1965 design had called for in setting crown heights for the southern reach of the IHNC.

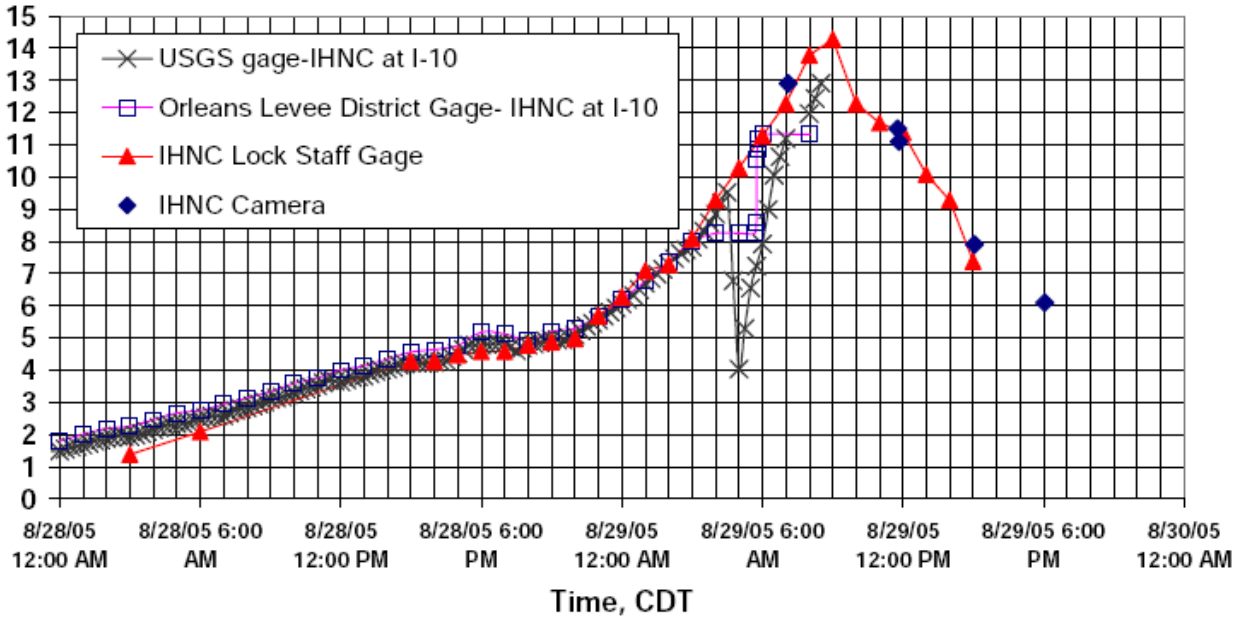


Figure 16. Gage records from IPET (2006b, Vol. IV-33)

Beginning a week after the storm, LSU researchers began acquiring high water marks (HWM) outside the HPS to verify the ADCIRC hindcast. Marks and debris lines on the unprotected side of the levees were all affected to some degree by waves, so water lines from inside buildings were sought to reduce these effects. Where no interior marks existed, debris lines on levees were included selectively, but always provide higher surge estimates than those acquired inside buildings. Fifty-two HWM elevations outside of the GNO HPS were surveyed.

Similar data was collected at 92 points inside the basins that flooded (Figure 17). Initially, these points were referenced to the statewide LIDAR dataset using a laser level (NAVD88). To reduce potential for later disagreement, LSU initiated collaboration with the IPET to locate and survey this perishable evidence. Eventually all HWMs outside the flood protection system were surveyed by either USACE, Louisiana Department of Transportation and Development (LDOTD) surveyors, or both (Figure 18). Local mean sea level, the reference for HPS design, varies from about 0.5 to 0.8 ft NAVD88 (2004.65).

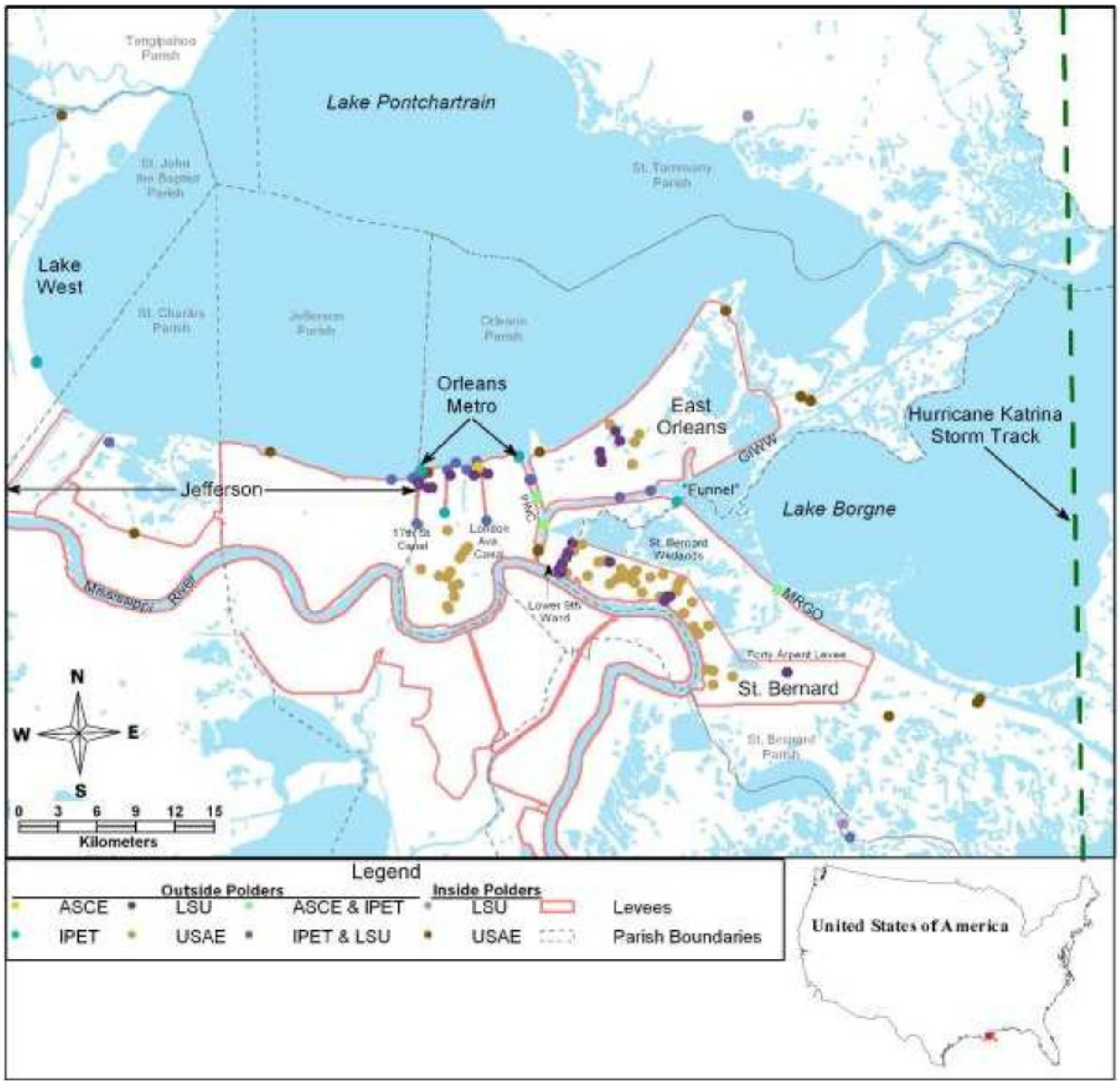


Figure 17. Greater New Orleans showing principal features that define flood defenses on the East bank; locations of high water marks surveyed outside and inside of the flooded basins, and five sub-areas used for reporting surge (Lake West, Jefferson, Orleans Metro, East Orleans and St. Bernard).

The marks surveyed within the flooded basins, include 58 in St. Bernard, 22 in New Orleans Metro, and 12 in New Orleans East. In addition to LSU and IPET, investigators from the USACE New Orleans District and the ASCE contributed to the HWM database. Peak elevations forecast by ADCIRC were converted from the NGVD29 ADCIRC datum to NAVD88 using the National Geodetic Survey VERTCON program, so that they could be compared to the HWM elevations (Table 2).

Zones West to East	No. HWM Obs.	Measured HWM (SE) feet (NAVD88)	Advisory 31 Hindcast
Lake West	5	***7.22 (0.69)***	***8.89***
		RMS Error	2.16
Jefferson	4	***8.04 (5.48)***	***12.23***
		RMS Error	1.12
Orleans Metro	20	***11.91 (1.02)***	***12.79***
		RMS Error	1.08
East Orleans	9	***13.71 (1.80)***	***13.71***
		RMS Error	2.07
St. Bernard	14	***15.94 (1.67)***	***13.78***
		RMS Error	2.36
All	52	***12.56 (0.46)***	***12.30***
		RMS Error	1.70
Percent Error	***52***		***14%***

Table 2.LSU ADCIRC Accuracy for Katrina outside the GNO Hurricane Protection System.

The LSU ADCIRC hindcast produced a mean elevation (12.3 ft) for the GNO only 0.3 ft lower than the observed mean, with a root mean square (RMS) error of 1.7 ft, for an average error of 14 percent. ADCIRC overpredicted surge to the west of the GNO and underpredicted to the east in the funnel area by about 2 ft, but was off by only about a foot on average along the GNO lakefront, and was nearly perfect at the entrance to the 17th Street Canal.

The LSU ADCIRC hindcast produced RMS error estimates of 1.1 to 2.4 ft for the three basins that experienced extensive flooding. Although efforts are made to minimize the effects, all HWMs from outside the levees contain an added, but unknown, increment of water level rise associated with waves. We believe that the best HWMs collected inside buildings include the surge and wave setup, but not the waves themselves, or run-up where the waves have already broken on levees or other structures. As a result, the difference between a mean surge elevation of 14.4 ft predicted by ADCIRC and 16.1 ft measured in a building just outside the St. Bernard levee system, for example, is at least partly attributable to waves.

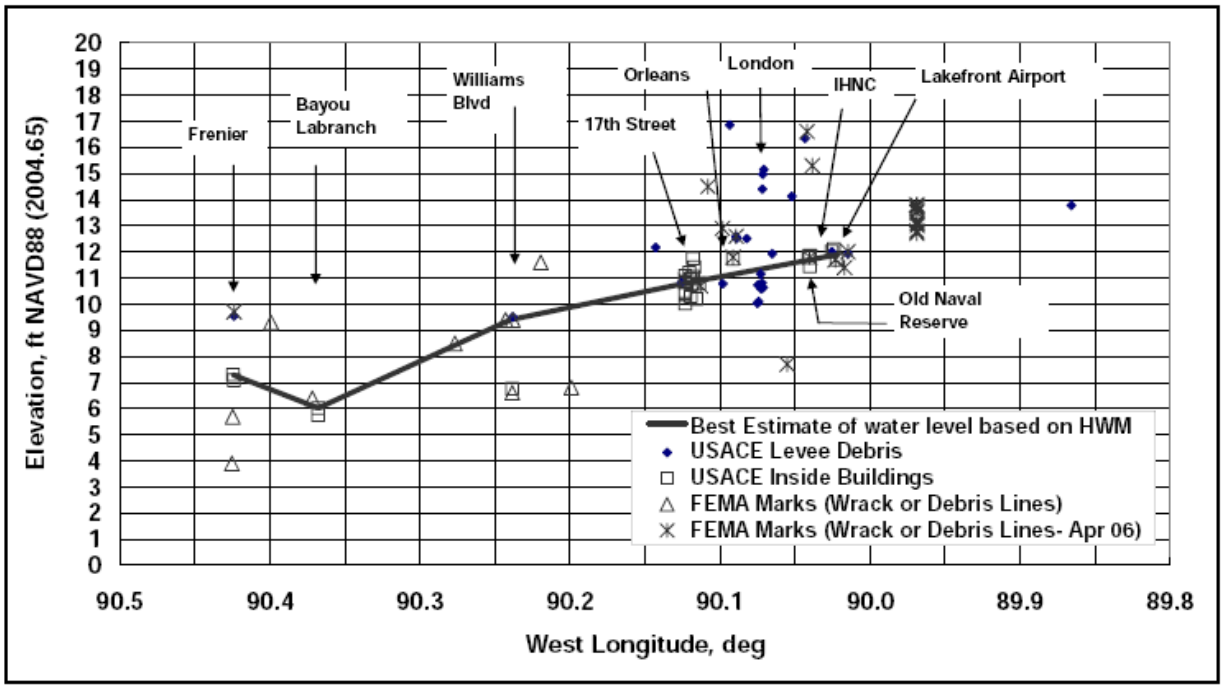


Figure 18. Plot of high water marks along the south shore of Lake Pontchartrain collected by LSU and other agencies, and surveyed by the USACE (IPET 2006b, IV-1-7).

Spatial trends in maximum surge predicted by ADCIRC were confirmed for both the Lake Pontchartrain and Lake Borgne surges. Along the south shore of the Lake, the surge increased from 6 ft to 13 ft going from Bayou LaBranche west of the City to Chef Menteur at the eastern margin of the lakeshore HPS (Figure 18). Waves added varying amounts of runup to different levee reaches. In the vicinity of the 17th Street and London Avenue Canal, the surge reached between 10 and 11 ft (NAVD88), but wave runup on some lake front levee faces added 5 ft (Figure 18).

The IPET analysis for the funnel area distinguishes between water level rise caused by surge and that attributable to waves. Test points were identified outside the New Orleans East and St. Bernard HPS (Figure 19). ADCIRC was used to identify trends in maximum surge elevation in the scatter created by the HWMs. Maximum surge elevation is estimated to range from 12 to 14 ft NAVD88(2004.65), increasing from north to south, or from Lake Pontchartrain towards the Lake Borgne funnel in the Chef Menteur area adjacent to the Bayou Sauvage National Wildlife Refuge on the east side of the New Orleans East HPS. The levee crown in the spots shown was within the design range of 13.5 to 15.5 ft MSL. Sea level in the funnel area is about 0.6 ft NAVD88(2004.65). Waves are believed to have added little more than an additional foot to the surge in this area, perhaps as a result of damping by the extensive wetland buffer in front of the levee. This levee did not experience general overtopping and came through the storm with little damage.

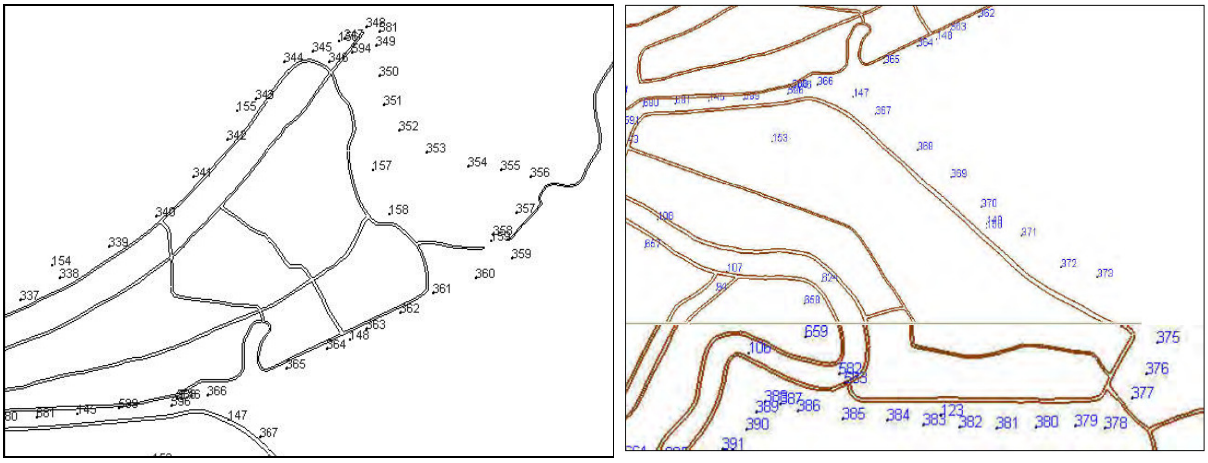


Figure 19. Test locations identified by IPET for analysis of surge and wave water levels in the funnel, for New Orleans East on left, and St. Bernard on right (from IPET 2006b, V-18-15, V-18-20)

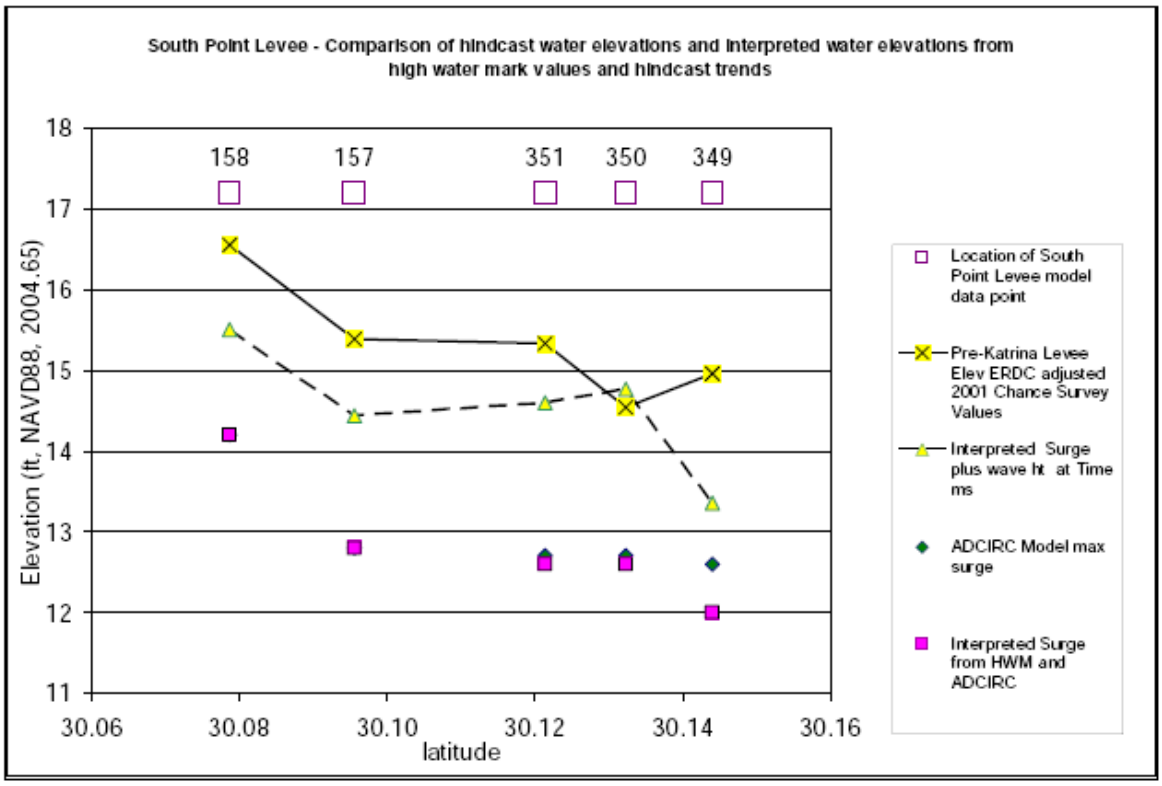


Figure 20. IPET estimates of surge and wave effects on east side of New Orleans East HPS. Surge ranges from 12 to 14 ft, with an additional rise of 1 to 2 ft due to waves or wave runup. Subtract 0.6 ft for elevation relative to sea level. Pre-Katrina levee in this reach was high enough to resist general overtopping (from IPET 2006b, V-18-19).

South of New Orleans East, the maximum surge is interpreted as rising from 14 to 16 ft MSL, increasing towards the GIWW/MRGO junction, but dropping by 1.5 ft in the throat of the funnel just west of Paris Rd (Figure 21). IPET added more

than a foot to the computed surge in this area to match the HWMs, and up to 2 ft more for waves, arriving at a combined surge/wave level of 16.5 to 18.2 ft MSL, somewhat below the design elevation of protection (17.5 to 23 ft MSL) but above the actual levee crown (15 to 17.5 ft MSL). IPET shows a potential for overtopping (0.5 to 2.0 ft) with depths similar to the crown elevation deficiency.

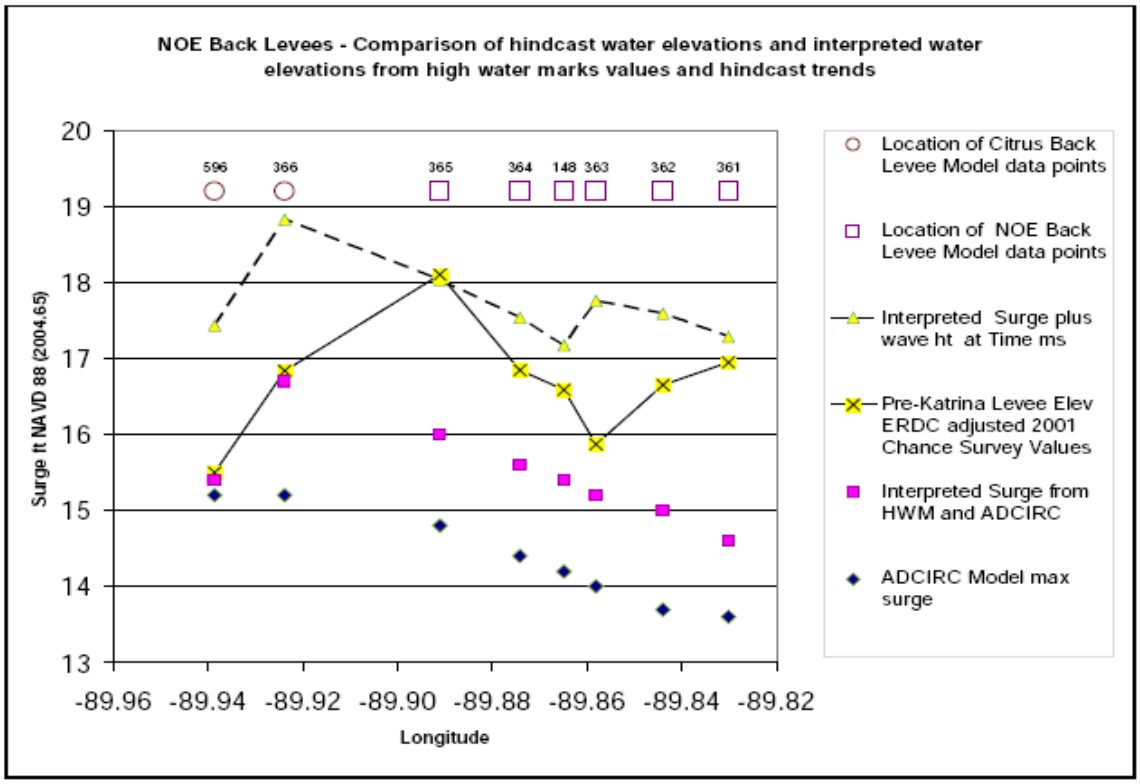


Figure 21. IPET estimates of surge and wave effects on north side of funnel, on New Orleans East HPS paralleling the GIWW. Surge ranges from 14.7 to 16.8 ft, with an additional 2 ft of rise due to waves or wave runup. Pre-Katrina levee in this reach was 15 to 18 ft high, but was subject to general overtopping (from IPET 2006b, V-18-18). Subtract 0.6 ft for elevation relative to sea level.

On the south side of the funnel along the MRGO, IPET interprets the maximum surge as being more uniform, ranging from 17.5 to 19.5 ft MSL (Figure 22). IPET had to add more than 2 ft to the computed surge to get to this level. Based on our understanding of the HWMs in this area, we would place the maximum surge at least a foot lower along the MRGO, but this is a matter for interpretation, and there is no right answer. IPET adds between 1 and 3 feet for waves, giving a combined water level that ranges from 19.5 to 23.0 ft MSL. The design elevation for the levee crown in this area is 17.5 ft MSL, and the actual elevation ranged from 15 to 18 ft MSL, so the stage was set for general overtopping. This overtopping, and the wave attack that preceded it, destroyed the MRGO levee, causing complete loss in some places and significant degradation in most (Figure 23).

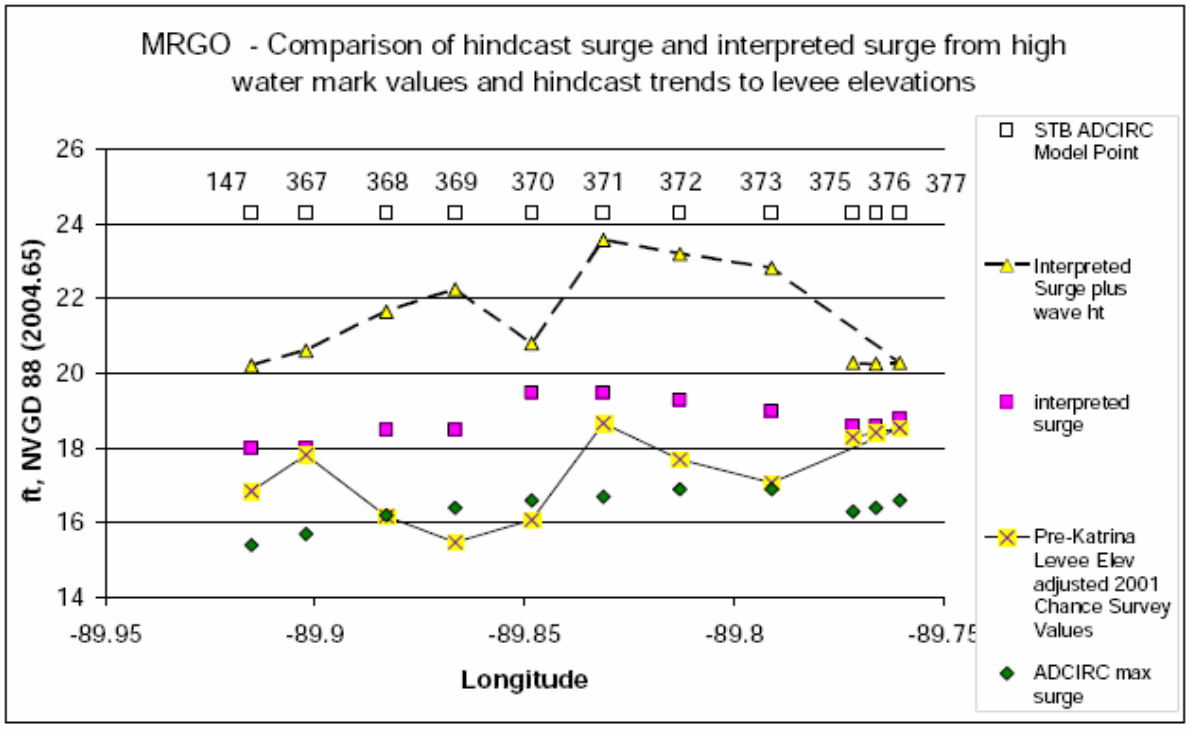


Figure 22. IPET estimates of surge and wave effects on south side of funnel, on St. Bernard (Chalmette) HPS paralleling the MRGO. Surge is estimated between 18 to 20 ft (NAVD88), with an additional 2 ft of rise due to waves or wave runup. Pre-Katrina levee was 15 to 18 ft high, but was subject to general overtopping (from IPET 2006b, V-18-24). Subtract 0.6 ft for elevation relative to current sea level.

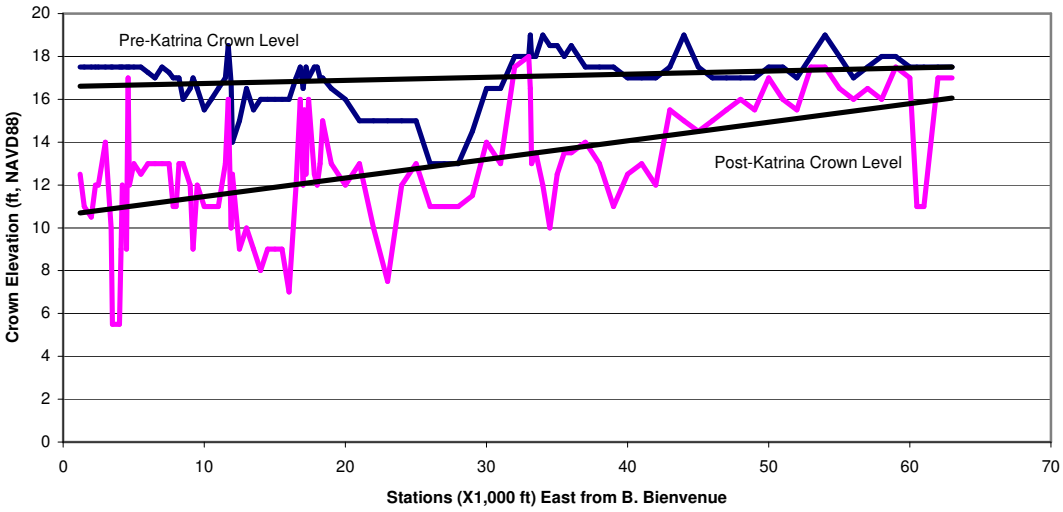


Figure 23. MRGO levee degradation from near junction with GIWW to Verret (derived from IPET 2006b, IV-16-2 to IV-16-10). Subtract 0.6 ft for MSL.

The IPET surge modeling team has made significant additions to the ADCIRC code and capabilities since the LSU hindcast on the day after the storm. The improvements include a more detailed representation of the Mississippi coast, use of reconstructed hurricane wind fields now available from the NHC, and addition of the physics of wave-induced surge amplification (wave set-up). The more recent IPET results improve understanding of the late-stage surge that propagated into eastern Lake Pontchartrain and the north shore (IPET 2006b). They also have produced better agreement with the surge observed west of the GNO in the LaBranche wetlands. But IPET has not improved the match to observed HWMs in the vicinity of the three GNO basins that flooded.

Given these advances, it may be serendipitous that the LSU results available a day after landfall resulted in surge estimates for most of the GNO that were as good or better than later IPET results, within 9 to 16 percent of the observed HWMs. The LSU predictions were based on the more intense Category 4 winds that were supplied by the NHC at the time the storm was active, before Katrina was downgraded to Category 3 at landfall (Knabb et al. 2005), and this probably compensated for the lack of wave set-up in the surge estimates.

Reconstruction of the Surge Sequence

ADCIRC results show not only where levees were challenged but also provide the sequence or 'when' of flooding. Snapshots of the predicted surge at hourly intervals beginning at 0700 on August 29 show the maximum migrating through the region (Figure 14). At 0700, the highest water is along the Plaquemines shoreline south of the GNO. At 0800, the surge is highest in the MRGO funnel. By 0900, surge is building along the south shore of Lake Pontchartrain. At 1000, the massive buildup along the Mississippi coast is apparent and water levels are dropping in the GNO.

Another way to look at this temporal sequence is through examining the record of water level at the same point through time. These are surge hydrographs (Figure 23). The model predicted that the Lake Borgne surge would peak first in the funnel on the MRGO and in the IHNC at about 0830, and at between 0900 and 0915 along the Lake Pontchartrain lakefront. The Lake Pontchartrain surge at the mouth of the 17th Street Canal was between 10 and 11 ft NAVD88(2004.65). Measured gage readings from the IHNC tend to confirm this sequence (Figure 16).

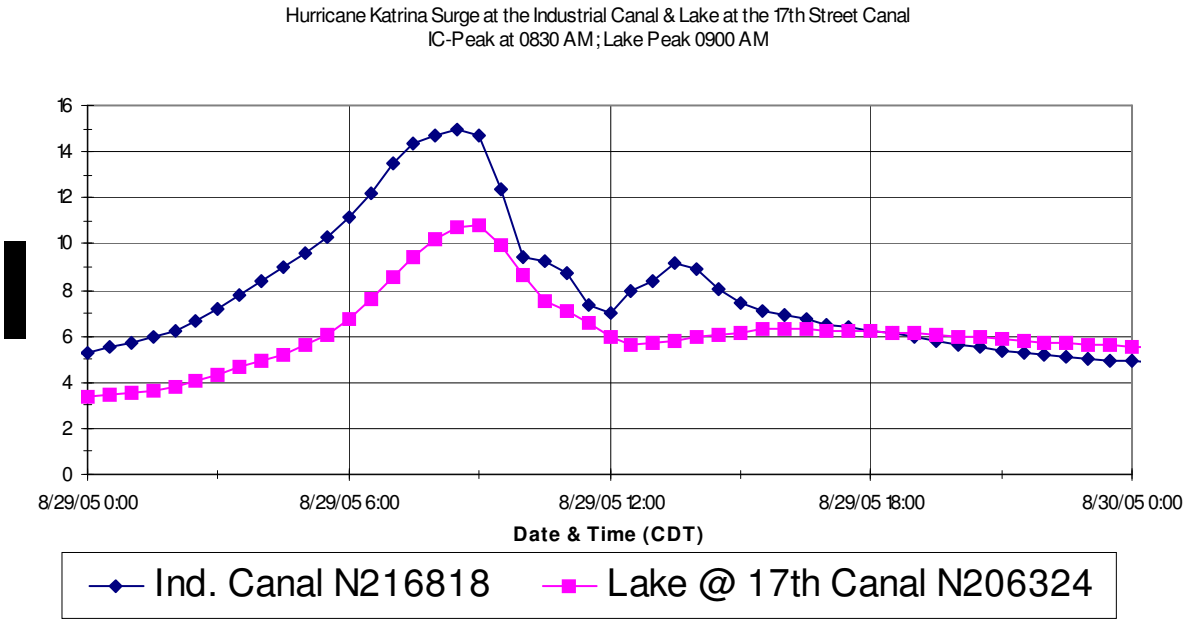


Figure 23. LSU ADCIRC model hydrographs for specific nodes in the IHNC (Industrial Canal) near the Lower 9th Ward and in Lake Pontchartrain at the 17th Street Canal.

Impact of Rainfall Runoff

IPET has tried to estimate the fraction of the total volume of water entering each of the flooded basins that could have been contributed by rainfall runoff that was not pumped (IPET 2006b, Vol. VI). They used two steady-state models, HEC-RAS (formerly HEC2) for channels, and HEC-HMS to convert precipitation to runoff, and route it through a variety of interior sub-basins. They used as-built levee and floodwall elevations to estimate overflow, and made assumptions about pump efficiency. The total volume of flooding determined by IPET did not differ much from our early estimates because they were using the same LIDAR base elevations and similar HWM data. For Orleans Metro, the IPET estimate of flooding from overtopping was slightly higher than an estimate we derived using ADCIRC (16 % versus 13 %), presumably because the actual floodwall elevations along the IHNC were lower than in ADCIRC. We attributed the remainder to the breaches on the IHNC and the drainage canals, but IPET divided the remaining floodwater volume (84 %) between that estimated to have been contributed by rainfall (26 %), and that entering via breaches (58 %).

Comparison of IPET and Team Louisiana Approaches to Rainfall

By focusing on the potential for rainfall flooding in the areas that did flood primarily as a result of flow through breaches, IPET seems to suggest that the GNO would have flooded without contributions from overtopping or the levee and floodwall failures. We do not consider runoff from precipitation to have played a role in determining the

maximum level of flooding in the Orleans Metro and St. Bernard basins for two reasons. First, flooding reached a level above mean sea level in these basins because of the **dynamics of flow through the breaches**. The level of the Lake remained elevated above the level of floodwaters within the Orleans Metro basin for more than two days after the breaches occurred, and inflow continued for this entire period (Figure 24).

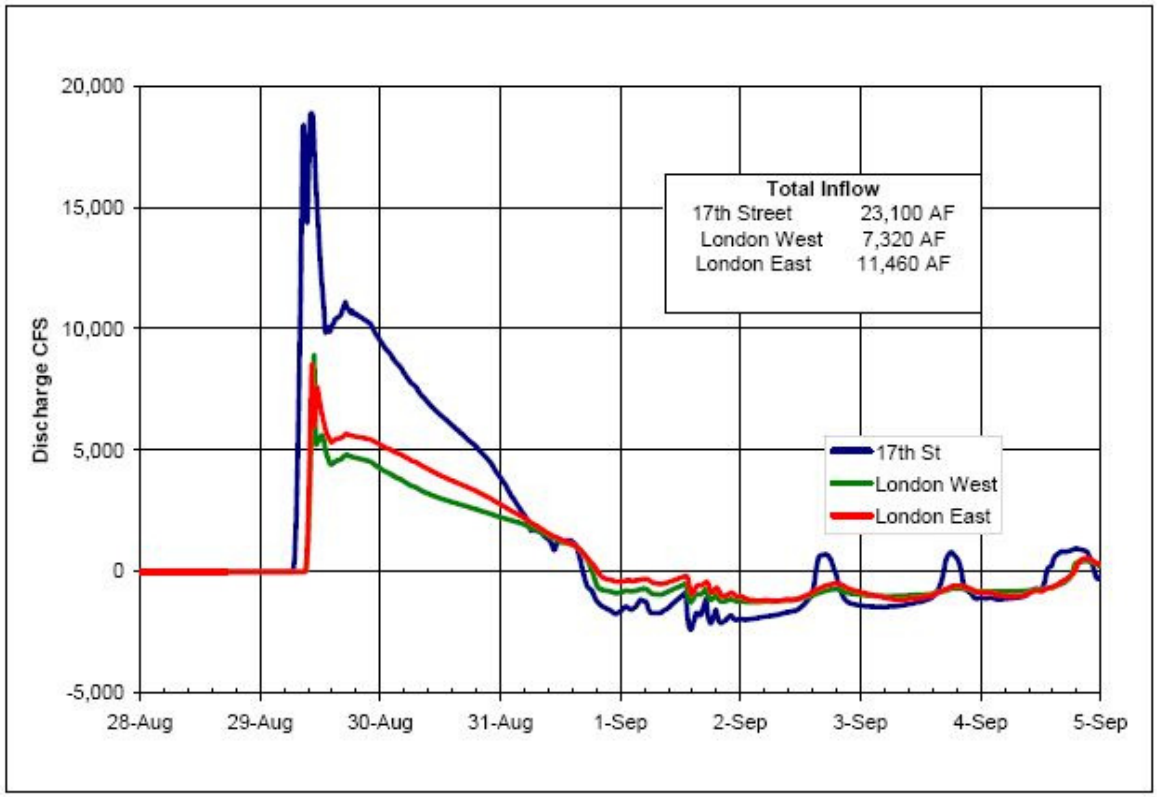


Figure 2-16. Calculated Flow Through Levee Breaches. Based on Observed Stage Hydrographs

Figure 24. Flows through Orleans Metro drainage canal breaches calculated by IPET (IPET 2006b, Vol. VI-2-16).

The ultimate elevation was determined by the head difference through time across the breach openings and would have been the virtually the same if no rainfall had occurred. Second, if breaches had not occurred in Orleans Metro that rendered the pumped drainage system inoperable and severely damaged, excess rainfall would have been removed within hours. The pumps failed because of the breaches, and would otherwise have removed excess runoff as quickly as they have for larger rainfall events in the past, like that associated with TS Allison in 2001, and the extratropical deluge that occurred in May, 1995.

One check on this analysis is the flooding that occurred in neighboring Jefferson Parish where no overtopping or breaching occurred. The staff from perimeter pump stations

were evacuated so that minimal pumping occurred in Jefferson as compared to Orleans Metro during the storm. While it has been suggested that backflow through the pumps from the Lake contributed to this flooding, IPET found that this was a relatively minor contributor (less than 5 %) as backflow prevention measures at most stations were quite effective (IPET 2006b, Vol. VI-1-8, Figure 1-4). The Jefferson basin is 4,000 acres larger than the Orleans Metro basin and was estimated by IPET to have generated a total runoff of 21,000 acre-ft. IPET estimated that the runoff in Orleans Metro, after subtracting 2,300 acre-ft that they estimated was pumped, was 22,000 acre-ft. The maximum flood elevation reached in Jefferson was between -3 to -4 ft (NAVD88) and resulted in mean flood depths of less than 2 ft in the lower-lying areas between the Lake and the Metairie ridge (IPET 2006b, Vol. VI-1-43, Figure 1-18). This was cleared within a day when the pumps were restarted. If the pumps had been operated through the storm, IPET determined that less than 10 percent of the area in Jefferson that flooded would have seen any accumulation of water (IPET 2006b, Vol. VI-1-42, Figure 1-17). If the Orleans Metro HPS had not been overtopped along the IHNC, and drainage canal I-walls had not breached, flooding should have been less extensive in Orleans Metro than in Jefferson, as the pump stations were occupied and would have operated through the storm.

Estimated Flooding Through Breaches

Katrina flooded about 84 percent of the developed portions of the three East Bank basins that experienced long-term flooding, ranging from 68 percent in Orleans East to 95 percent in St. Bernard. The overtopping predicted by ADCIRC across idealized levee and floodwall crowns showed far less water getting into New Orleans than actually entered the city, ranging from an estimated 11 percent of the flood volume in St. Bernard to 33 percent in Orleans East. Only the lowest 34 percent of the nearly 55,000 acres that were submerged on 29 August would have experienced standing water once the storm passed. Moreover, mean flood depths caused by overtopping alone would have been less than 2.3 ft in all three basins. IPET predicted higher values for overtopping in St. Bernard and New Orleans East because they used actual levee heights that were substantially lower in places than the design level of protection (Figure 25).

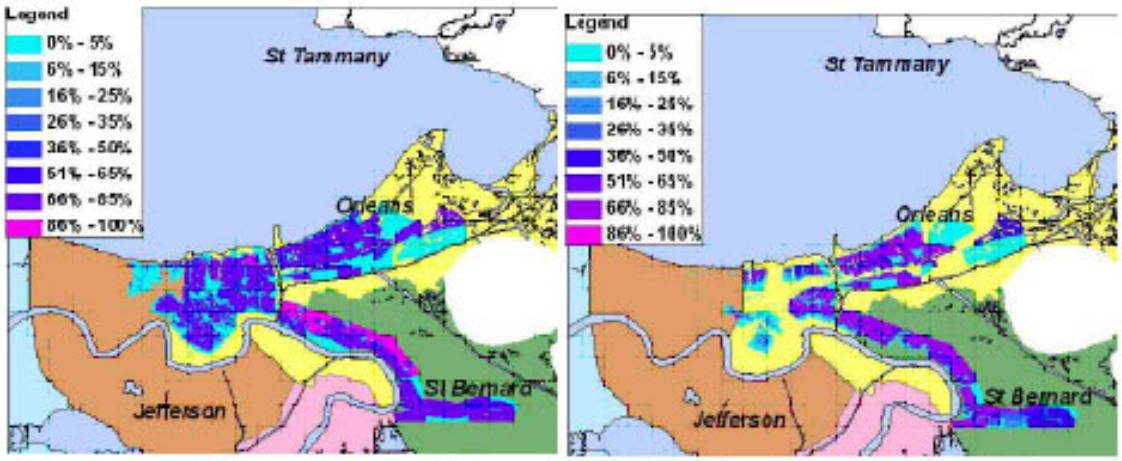
The surge predicted by ADCIRC should have exceeded the levee crown elevations claimed for the HPS by no more than 3 ft anywhere in the GNO for less than two hours. An overtopping event of this type would not have caused the extensive damage to interior drainage and pumping infrastructure that, in fact, took place. Because they determined how high the water got, we believe breaches were responsible for nearly 90 percent of the flood volume in Orleans Metro and St. Bernard. They were least significant in Orleans East, perhaps as low as 17 percent according to IPET, where the

flood elevation remained below sea level, and overtopping and rainfall played greater roles.

Comparison of Estimated Mean Losses from Katrina with Hypothetical Scenarios ($millions 2005)

Basin Name	Katrina Model		Hypothetical Katrina Scenario #1 (Resilient Levees)		Hypothetical Katrina Scenario #2 (Resilient Levees and Pumps)		Hypothetical Katrina Scenario #3 (Resilient Floodwalls)	
	Water Surface Elevation	Property Loss Estimate ($millions 2005)	Water Surface Elevation	Property Loss Estimate ($millions 2005)	Water Surface Elevation	Property Loss Estimate ($millions 2005)	Water Surface Elevation	Property Loss Estimate ($millions 2005)
JE1	-4.10	428.8	-4.10	428.8	-7.00	8.1	-4.10	428.8
NOE1	3.80	10.2	2.90	9.0	1.90	7.8	3.80	10.2
NOE2	-0.30	115.7	-1.00	116.3	-4.90	2.4	-0.30	115.7
NOE3	1.70	481.9	-2.90	54.6	-0.80	379.4	1.70	481.9
NOE4	7.60	57.1	7.10	56.7	7.00	56.6	7.60	57.1
NOE5	-0.80	4,636.6	-1.80	4,209.0	-3.90	3,123.5	-0.80	4,636.6
OM1	2.60	2,130.2	-0.90	1,695.0	-5.10	132.7	0.0	1,713.2
OM2	3.20	1,595.7	-2.50	962.1	-5.00	677.1	-2.70	930.9
OM3	3.80	1,965.8	3.10	1,740.8	2.90	1,674.3	3.80	1,965.8
OM4	2.30	341.8	0.10	149.8	-1.50	48.1	0.10	149.8
OM5	2.60	3,376.8	-0.80	924.6	-2.00	785.0	-0.40	1,203.9
SB1	10.50	2,562.1	4.20	1,774.9	3.80	1,700.2	10.50	2,562.1
SB3	10.90	2,383.9	3.70	1,412.1	3.70	1,412.1	10.90	2,383.9
SB4	11.20	492.3	6.60	253.3	6.40	232.2	11.20	492.3
Total		20,579.0		13,787.0		10,291.4		17,182.3

Note: The water surface elevation for JE2 is set to produce flood damages of zero assuming that the pumps could evacuate the rainwater. Does not include infrastructure losses.



Percent of Property Damaged (Damage/Value)–Comparison of Model Results for Katrina Flooding (left) and Hypothetical Scenario #2, Resilient Levees and Pumps (right)

Figure 25. Location of damages calculated by IPET (2006b, VII-30) on right if the levees and floodwalls had not breached. Damage estimate with breaches is $20.5 billion, and without breaches, $10.2 billion. IPET estimates that if the levees and floodwalls had not failed (were 'resilient') the damages would have been about $3 billion for each of the three flooded basins of Orleans Metro, New Orleans East and St. Bernard. Actual damages associated with Katrina exceeded $100 million.

IPET estimated that of the water introduced into the Orleans Metro over and through the HPS, 37 percent came through the 17th St. Canal breach and 30 percent through the two London Ave. Canal breaches. About half of the remainder (32 %) entered over the low floodwalls and levees along the west side of the IHNC, while the rest came through a levee breach south of the Gentilly ridge at the Port of New Orleans container facility (IPET 2006b, Vol. VI-2-42, Table 2-12). Processes that led to the initiation and expansion of breaches, particularly deficiencies in the elevations of some levee and floodwall reaches greatly influenced the volume of water introduced into each basin. Large breaches that happened early resulted in greater depths of flooding.

The east side of the GNO that faces Lake Borgne experienced a higher level of storm surge during Katrina than the Orleans Metro south of Lake Pontchartrain (Figure 26).

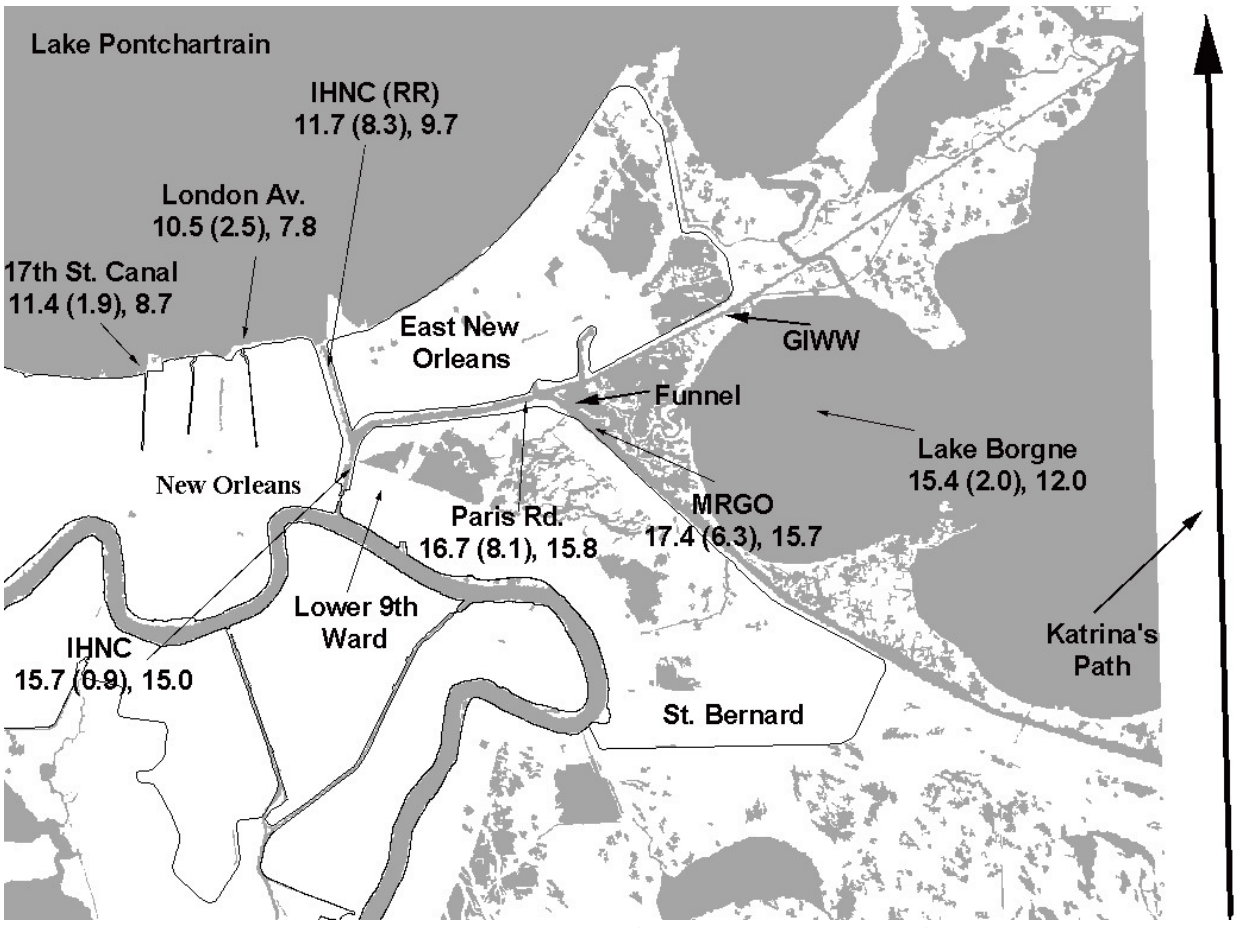


Figure 26. LSU ADCIRC maximum surge elevation (ft) and current velocity (fps) in parentheses based on the LSU hindcast. The arrow indicates the point at which the surge and velocity is reported. The final number is the surge elevation (ft) forecast 39 hours prior to landfall. (Mashriqui etal 2006)

The surge from Lake Borgne propagated as a high velocity bore through the MRGO and GIWW to the IHNC causing maximum water levels in that canal to rise to 15 ft NAVD88(2004.65), while surge along the lakefront to the west was 2 to 4 ft lower. The highest surges predicted and observed for the greater New Orleans area were in the throat of the funnel east of Paris Road. Overtopping of levees in the MRGO connection to the IHNC west of Paris Road is believed to have provided the majority of the water that flooded New Orleans East.

The MRGO and GIWW levees east of the junction are the only GNO HPS levees that should have experienced extensive overtopping during Katrina. Unfortunately for St. Bernard, the MRGO levee did not wait for overtopping, but was largely destroyed before it could be overtopped, at an earlier stage in the storm sequence, as is discussed in the next section.

CHAPTER THREE

THE FLOODING OF NEW ORLEANS

Sources of Information on Flooding Sequence

The ADCIRC surge model provided an accurate overview of the sequence of stresses affecting the GNO HPS. The USACE has identified 50 locations on the GNO HPS where breaches occurred as a result of (1) structural instability, (2) overtopping and erosion, or (3) some combination of the two. Verifying the timing of levee breaching or overtopping was not easy despite a number of reports from those who claimed to witness the onset of the flooding. Witnesses, themselves under great duress, tended to offer differing versions of the same events and particularly could not agree on the timing. Heavy rainfall kept witnesses indoors except during lulls, and wind-driven spray from breaking waves obscured the view from high rise buildings along the lakefront.

For more than a month after the storm, the chronology continued to be debated. Ultimately, we were able to collect reliable reports from radio logs of emergency personnel and from cell phone records as they became available, and combine these with stopped clock data. Obviously it was important to understand exactly when the breaches occurred, but another purpose was to establish the sequence and rapidity of flooding. An additional objective was to assess whether the local nature and rate of flooding affected the loss of life. It is now understood that these variables differed widely across the neighborhoods inundated (see Appendix 3).

An article in the New Orleans Times-Picayune in mid-October 2005 provided affected residents with a telephone number to report stopped clock data from their flooded homes to help LSU investigators support a timeline for flooding. As we hoped, this approach was picked up by IPET investigators. IPET had the manpower and resources to interview over 600 people and survey a large number of stopped clocks. Very importantly, IPET was able to develop hydrographs based on sequences of time/date stamped digital photographs. Most of the information developed from these investigations did not become available, however, until June 2006, when IPET issued its final report.

The Stopped Clock Data

Nearly fifty stopped clock times were reported to the LSU Hurricane Center and field investigators during October and November as residents were returning to their homes.

Some sent photographs. Battery-operated clocks were the source of most information, but grandfather clocks with pendulums were also included in the database (Figure 27). Numerous times were also submitted by eye witnesses reporting when they first noticed ponding, especially those who used cell phones to report the flooding. Cell calls are date- and time-stamped and this information could be obtained from billing records. All reported clock times were documented in a phone log or field list and entered into a spreadsheet indicating type of clock, address, zip code, and latitudes/longitudes where available. For some, additional information was given on height of clocks, which ranged from three to ten feet above ground, or other supplemental data (e.g., slab elevation). Efforts were made to verify reports and to encourage citizens to also report the information to USACE IPET investigators, when they began to look for and examine stopped clocks.

Addresses of stopped clocks and times were imported into ArcGIS 9.1, geocoded and mapped using 2004 Tiger Roads data. Clock times were displayed for further analysis along with map layers such as levee shapefiles (with location of 17th Street Canal and two London Ave. canal breaches indicated), roads, extent of flooding on August 31, 2005 (NOAA data) and georeferenced Ikonos imagery (September 1, 2005) supplied by the LSU Coastal Studies Institute (Appendix 2). The great majority of stopped clock reports came from the Lakeview neighborhood that was flooded primarily by the 17th Street Canal breach. Fewer are available for each of the London Avenue Canal breaches, and for the Lower 9th Ward where the damage to homes was more extensive.

Figure 27. An example of a flooded battery-operated clock from Gentilly.

The stopped clock data must be used with care, and were generally used as supporting data that could assist in corroborating eyewitness reports and provide clues about the rate and spatial evolution of flooding. First, stopped clock times do not indicate AM or PM and therefore judgment is needed to decide which of two times separated by 12 hours is most likely and fits in best with other reports. Second, while some reported clock times were verified with photos, a number of clock times and reports of first flooding that were reported could not be verified. Third, slight time variations exist and must be accounted for, in that eye witness accounts generally reported times when ponding was first noticed, while battery operated clocks, typically wall mounted kitchen clocks, shorted out when water reached the batteries. Grandfather clock times were likely reflective of when floodwaters reached the pendulum. Fourth, it cannot be assumed that all clocks that stopped were actually set to the proper time.

With these caveats stated, it is still possible to glean good data from the stopped clocks that could be augmented with first responder call logs, cell phone records and limited video photography (date/time stamped). The clock data from the Lakeview neighborhood gives an indication of the complexity of the flooding event caused by the 17th Street Canal breach. As time passes, flooding can no longer be strictly linked to any single breach as the influences of individual breaches merge.

Clocks, particularly pendulum clocks, in the immediate vicinity of the 17th street breach give a "no later than" indication of when the breach occurred. The earliest stopped clock was a pendulum clock located about half a mile from the breach that stopped at 10:07 (Figure 28). The owner assures us that it was keeping time accurately before the storm. The pendulum on these clocks stopped when water got to 2 to 3 ft above ground elevation in slab-on-grade homes. Other grandfather clocks farther away from the breach stopped later. A pendulum clock located a mile from the breach stopped 2.5 hours later, but still earlier than most wall-mounted electronic clocks. Most wall-mounted electronic clocks are 7 to 9 ft above ground elevation. They generally stopped 7 to 11 hours after the breach and the times showed no relationship with distance from the breach. This suggests that outside the immediate vicinity of the breach, water rose fairly slowly and uniformly as the bowl filled.(www.publichealth.hurricane.lsu.edu/delft) The variation in time to stopping reflected simply the variability in clock elevations. There were two exceptions to this rule. First, clocks between the 17th Street Canal and the elevated West End Boulevard a few blocks to the east tended to see a higher rate of rise (stopped more quickly) if they were south of the breach. The elevated boulevard divides the basin into smaller sections so that the section closer to the breach filled first before cascading into the larger main bowl to the east. Also, because the area to the north of the breach is built up on fill from the lake, wall clocks in homes located quite close to the breach but on higher land to the north were flooded later.

Figure 28. An example of a pendulum clock that stopped at 10:07.

Overtopping and Breaches

The breaches are discussed in chronological order starting with the first breaches before Katrina's landfall. Based on phoned-in reports, it is likely that the first flooding inside the GNO HPS on the East Bank occurred from the Mississippi River in the Poydras area of St. Bernard Parish (Figure 29). Most of the water that came across the levee was probably wave-driven, because the surge on the River did not peak until about 0800 on the Carrollton Gage (Figure 30). Water was observed rushing down streets shortly after daylight, about the time of the Louisiana landfall, but this apparently ceased after the water "came through." Such a report would be consistent with limited wave-driven overtopping from the Mississippi River, which experienced a 13 ft bore and upstream flow as the hurricane crossed the mouth, tossing barges and ships onto the levee slopes (Figure 31).

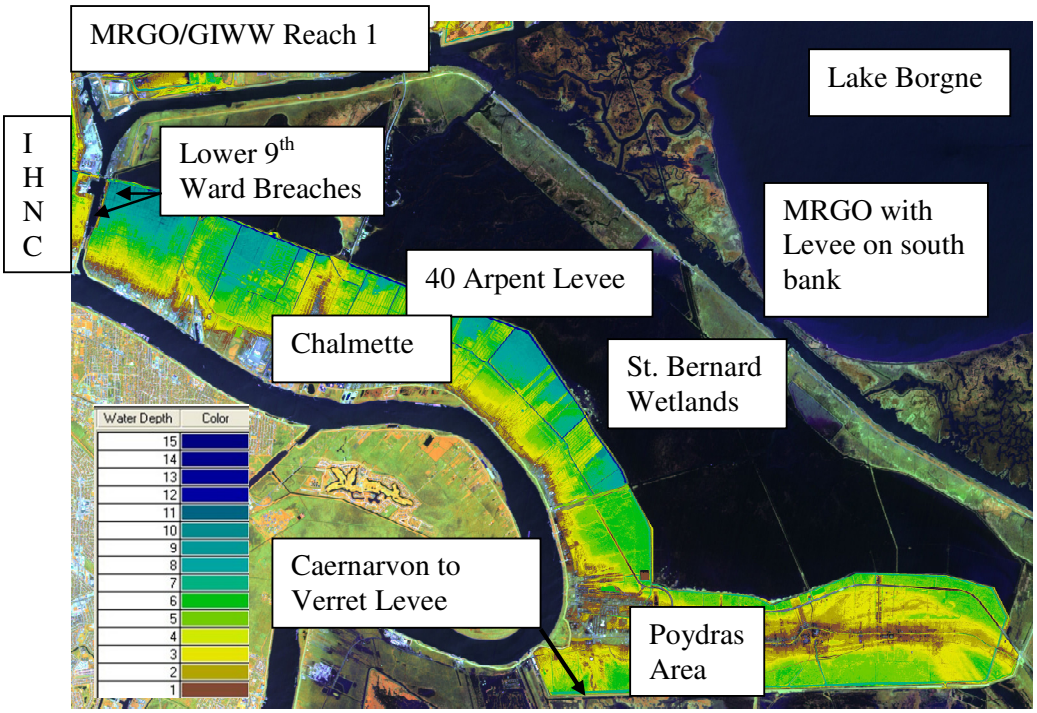


Figure 29. The St. Bernard HPS, showing elevations of the developed area, with the 5 ft contour in brown, and yellow, green and blue lower. The federal HPS follows the IHNC and the MRGO on the west and north sides, and ties back into the mainline Mississippi River levee on the south side at Caernarvon. Poydras reported first Katrina flooding from Mississippi River on East Bank. More wetland than developed area is included within the area protected by the federal HPS. A local levee, the 40 Arpent Levee, was built by the State and St. Bernard Parish to allow pumped drainage of the developed area inside the federal HPS.

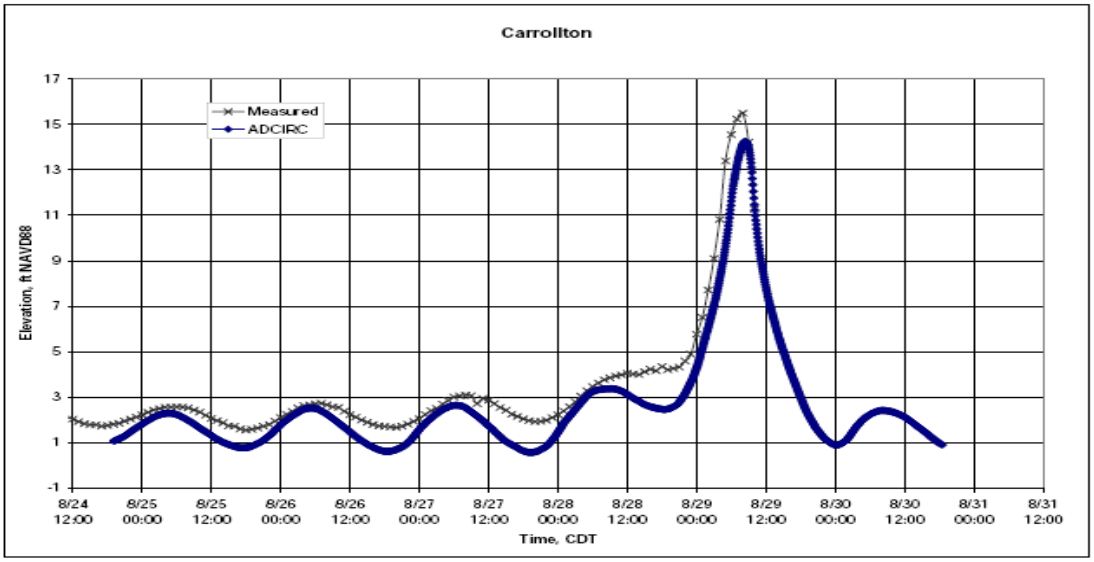


Figure 30. Surge on Mississippi River at the foot of Carrollton Avenue in New Orleans peaked above 15 ft at 0800, 29 August, 2005. The thinner, darker line is the actual gage reading, while the blue is an IPET ADCIRC result (IPET 2006b, IV-5-67).

Figure 31. Looking downstream from Gretna at ferries and barges on both banks of the Mississippi River batture and levee.

Early Flooding on the IHNC

The first sustained flooding of GNO residential areas occurred well before Katrina made landfall in Louisiana. Between 4:30 and 5:00 am, water began to come through a gap on the west bank of the IHNC where the CSX Railroad crosses the northern arm of the canal, adjacent, and parallel to, the Interstate 10 bridge (Figure 32). Here, sand bags had been laid across the tracks (Figure 33). The metal gate that should have sealed this opening had been damaged during a train derailment, and had been removed for repair. This water flowed into the Desire or Upper 9th Ward neighborhood (Figure 34) when a nearby water level recorder on the IHNC recorded the surge was 9 feet (NAVD88) (Table 3a). Temporary protection on the east side of the canal also apparently blew out about this time initiating the first flooding of the Orleans East Polder.

A larger breach developed south of the railroad crossing, also on the west side of the IHNC, near the Florida Avenue Bridge at the France Road terminal (Figure 35) at about the same time (Table 3a). This was the major breach that flooded the eastern part of Orleans Metro south of the Metairie Ridge (Figure 36). In one location, the wall failure caused erosion of a large scour hole in an earth levee composed primarily of sand and shell.

Table 3a Significant Breach Sources of Katrina Floodwater in GNO HPS							
HPS Zone	**Breach Name**	**Type**	**Cause**	**Top Width (ft)**	**Sill Elev (ft)**	**TimeStart**	**Start Water Level**
Orleans Metro							
Lakeview	17th St. RELee	I-Wall	Translation Slide	450	0.0	6:00-7:00	6.5
Fillmore	London North	I-Wall	Underseepage/ Rotation Slide	180	0.0	7:00-7:30	8.2
Mirabeau/ Dillard	London South	I-Wall	Underseepage	100	-4.0	6:00-7:00	7.1
City Park	Orleans Canal	Earth Levee	No Floodwall Installed	150	9.5	8:30	
Desire	IHNC West RR	RR Gate Missing	Sandbag Plug Blowout	37	5.1	4:30	9.0
Desire	IHNC West Florida Ave. North Breach	I-Wall	Overtopping Erosion	100	7.0	7:00	12.0
Desire	IHNC West Florida Ave. South Breach	Earth Levee	Overtopping Erosion	125	5.0	5:00	10.0
St. Bernard/ Lower 9th Ward							
Lower 9th Ward	IHNC East Bank North	I-Wall	Overturned/ Insufficient Levee	90	7.0	7:30	12.5
Lower 9th Ward	IHNC East Bank South	I-Wall	Overtopping Erosion Underseepage	1,000	5.0	7:30	12.5
Chalmette, Poydras, Verret	MRGO	Earth Levee	Wave Erosion Overtopping	60,000	0.0	5:00	12.0
Lower 9th Ward	IHNC/ GIWW	Wall Levee	Transitional Failure	30	4.0	6:00	11.0

Table 3b (cont'd). Significant Breach Sources of Katrina Floodwater in GNO HPS							
HPS Zone	**Breach Name**	**Type**	**Cause**	**Top Width (ft)**	**Sill Elev (ft)**	**TimeStart**	**Start Water Level**
New Orleans East							
B. Sauvage NWR	NOEast Back (1)	Earth Levee	Overtopping Erosion	5,000	10.0	5:00	12.0
B. Sauvage NWR	NOEast Back (2)	Earth Levee	Overtopping Erosion	1,200	9.5	5:00	12.0
B. Sauvage NWR	NOEast Back (3)	Earth Levee	Overtopping Erosion	800	10.5	5:00	12.0
B. Sauvage NWR	NOEast Back (4)	Earth Levee	Overtopping Erosion	139	5.5	5:00	12.0
Read Blvd E.	Citrus Back (1)	Earth Levee	Overtopping Erosion	480	11.0	5:00	13.0
Read Blvd E.	Citrus Back (2)	Earth Levee	Overtopping Erosion	550	8.5	5:00	13.0
Read Blvd W.	Citrus Back 3)	Earth Levee	Overtopping Erosion	380	11.0	5:00	13.0
Jourdan/Almonaster	INHC East RR	RR Gate Missing	Sandbag plug blowout	37	5.5	4:30	9.0

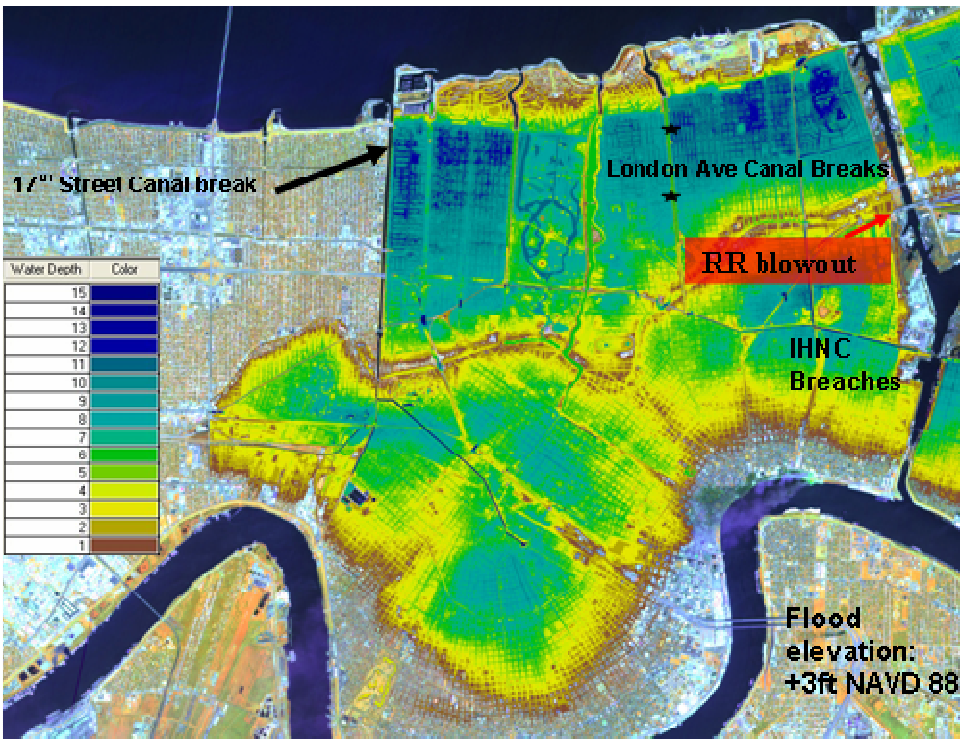


Figure 32. Orleans Metro Bowl showing locations of 17th Street, London Avenue and IHNC breaches, from SPOT satellite image on September 2, 2005, with water depth overlay from 1 (brown) to greater than 12 ft (dark blue).

Figure 33. Looking northeast at sandbag blowout on RR penetration of I-Wall on west side of the IHNC near I-10 crossing.

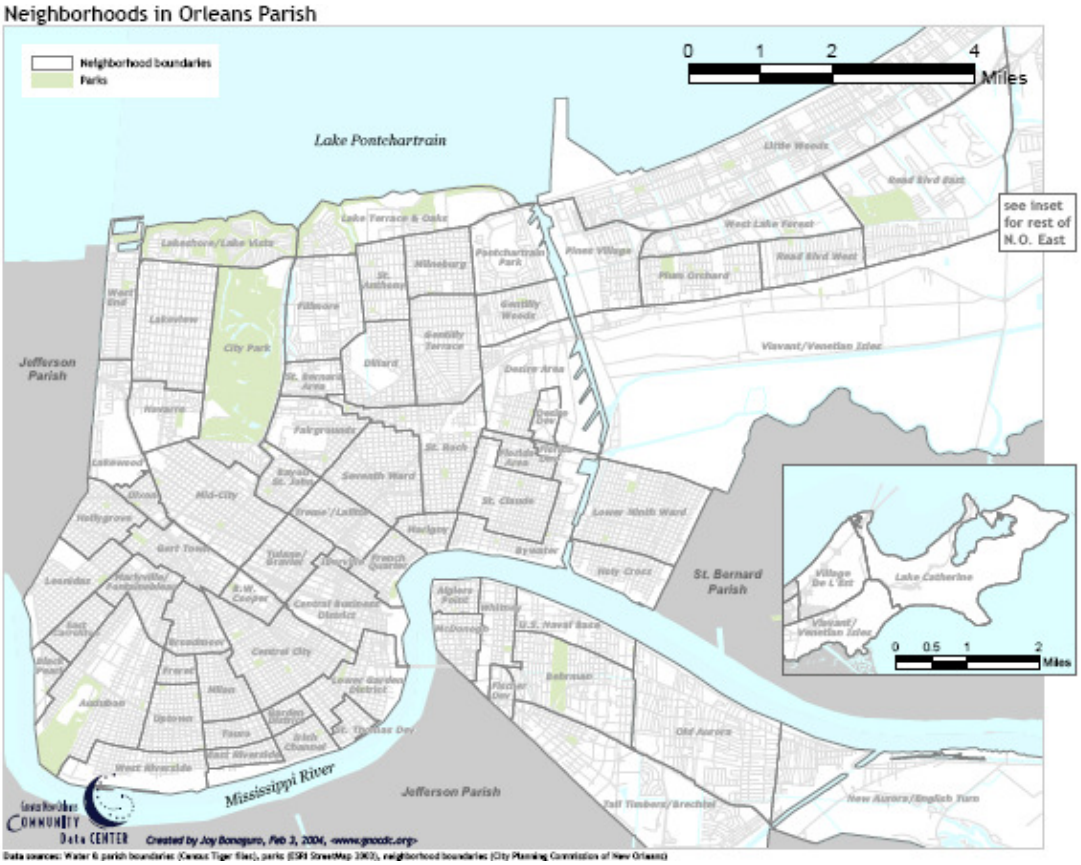
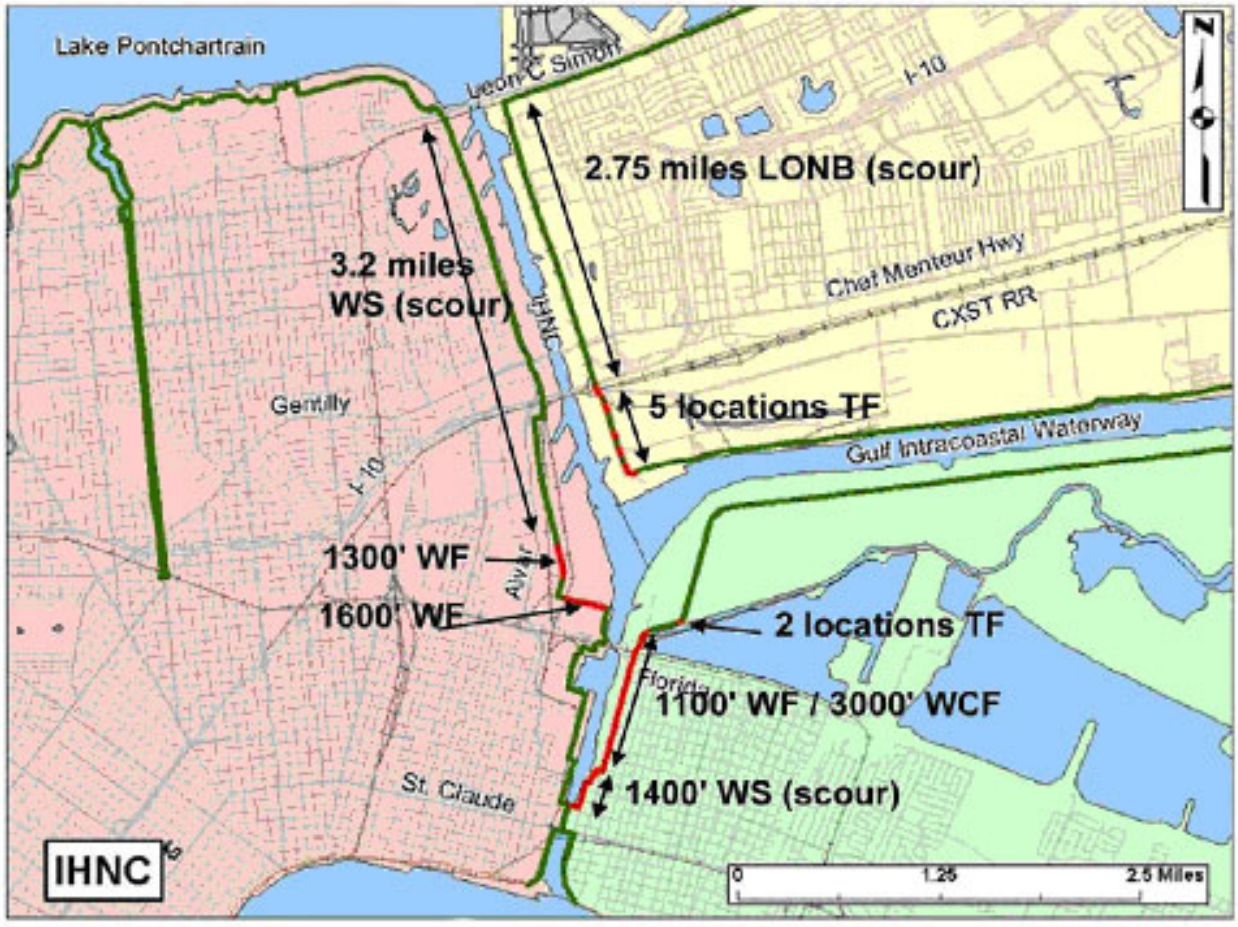


Figure 34. Neighborhoods of New Orleans courtesy of Greater New Orleans Community Data Center (www.gnocdc.org)

Ninety minutes later (6:30 AM), residents of the Lower 9th Ward on Tennessee Street saw water coming up the street from the east. It was a "slow" flow so they weren't sure if it was just rainwater. We are not sure of this source but it most likely represented some potential overtopping of sections of the levees along the southern side of the GIWW between its junction with MRGO and the 'T' into the industrial canal. Eye witness reports are that this levee was submerged by 6:00 AM

IPET (2006b) has reported that the northern failure of the I-wall fronting the lower 9th ward occurred around 6:00 AM, however, as will be discussed shortly neither ILIT or Team Louisiana agree with this IPET assumption. This breach eventually attained a width of about 90 ft as flow through it increased while the water continued to rise to more than 14 ft (NAVD88).



IHNC Flood Protection Damage

Figure 35. Overtopping and breaching along the IHNC. IPET developed the terminology shown above to distinguish failure modes, and cause of flooding (IPET 2006b, V-14-18) Damaged areas along the IHNC: LONB= Overtopped levees, no breaching, WS=Overtopped floodwalls, no breaching(stable), LOB=Overtopped levees, breaching, TF= Transition failure (floodwall to levee transition), WF= Overtopped floodwalls, breached(failure)

Figure 36. Looking southwest at first major breach into Orleans Metro on west side of IHNC at transition between floodwall and earth levee at container facility.

Figure 37. Looking north at one of two breaches from IHNC east into the Lower Ninth Ward, known as the north breach. I-wall remains inverted where it was overturned.

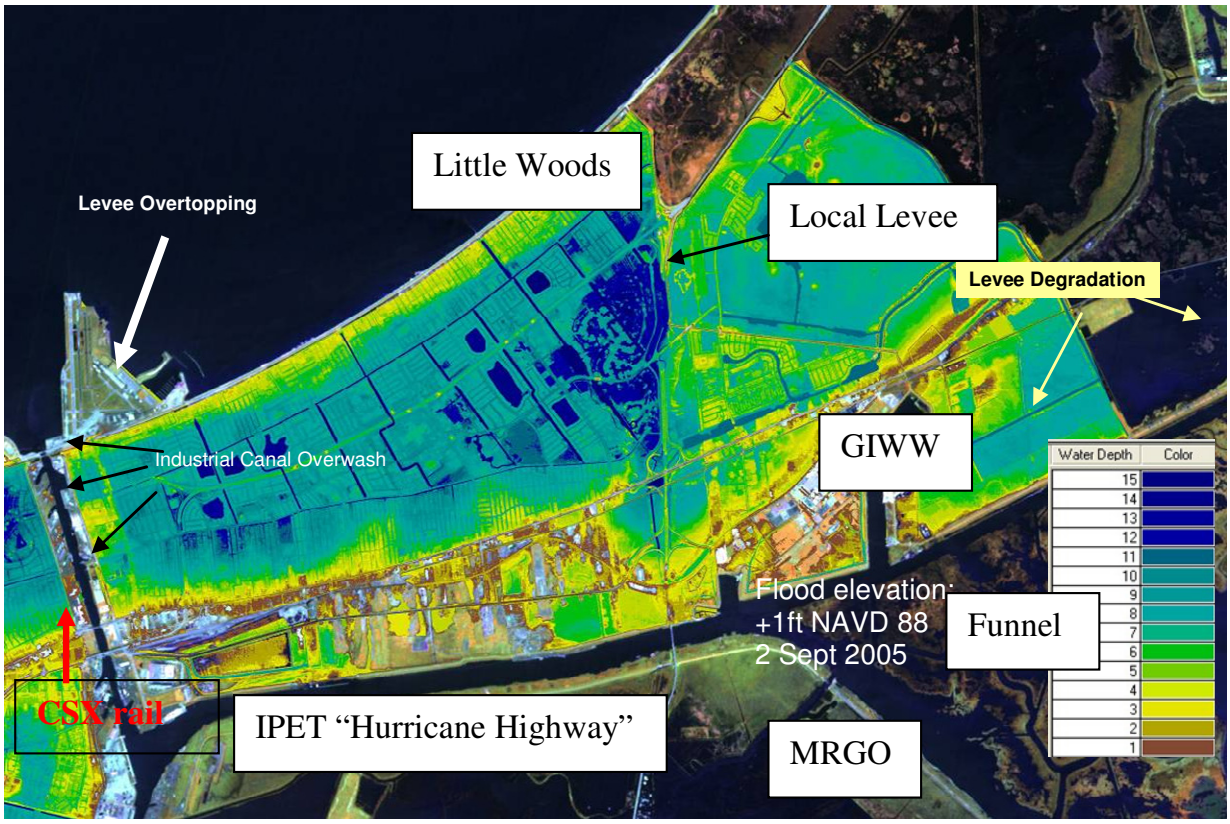


Figure 38. New Orleans East showing areas of levee degradation and breaching, as well as flood depths 4 days after the storm. Water overtopping the GIWW and MRGO levees flowed over higher industrial property adjacent to the channel, across the Chef Menteur ridge and CSX railroad embankment before reaching residential area to the north.

Early Flooding in the Funnel

The geometry of what has become known as the MRGO funnel is formed by the convergence of levees that follow the GIWW and MRGO channels, open on the east to Breton Sound, partially enclosing Lake Borgne, and narrowing to a 1,500 ft wide, 40+ foot deep ship channel west of the junction of the MRGO and GIWW (Figure 38). The 6-mile portion of the combined MRGO/GIWW that connects the funnel with the IHNC (IPET "MRGO Reach 1") is what IPET has called the hydraulically critical reach, though the USACE apparently still objects to the "hurricane highway monicker," as they called it (IPET 2006b, I-6). Team Louisiana has used the term "Hurricane Alley" for this reach. The USACE said much the same after Hurricane Betsy flooded many of the same areas in 1965 (Shallat 2000). It was this channel reach that added 3 ft to the surge in the IHNC that would otherwise have mirrored Lake Pontchartrain, and raised water levels so quickly that it caused IHNC breaches before the hurricane made landfall (Figure 39).

In the 0500 to 0600 period, still prior to landfall, the Katrina surge rose above 12 ft (Figure 40) on the New Orleans East Back Levee that follows the GIWW and faces south

toward Lake Borgne. This levee experienced significant wave attack (Figure 39). IPET reports that it was built of erosive sandy material dredged from the GIWW and was severely degraded in numerous places. The areas protected behind these levees are largely undeveloped, however, and these wetlands provided some storage before a lower, local levee (Maxcent Levee) was overtopped on the back side (Figure 39).

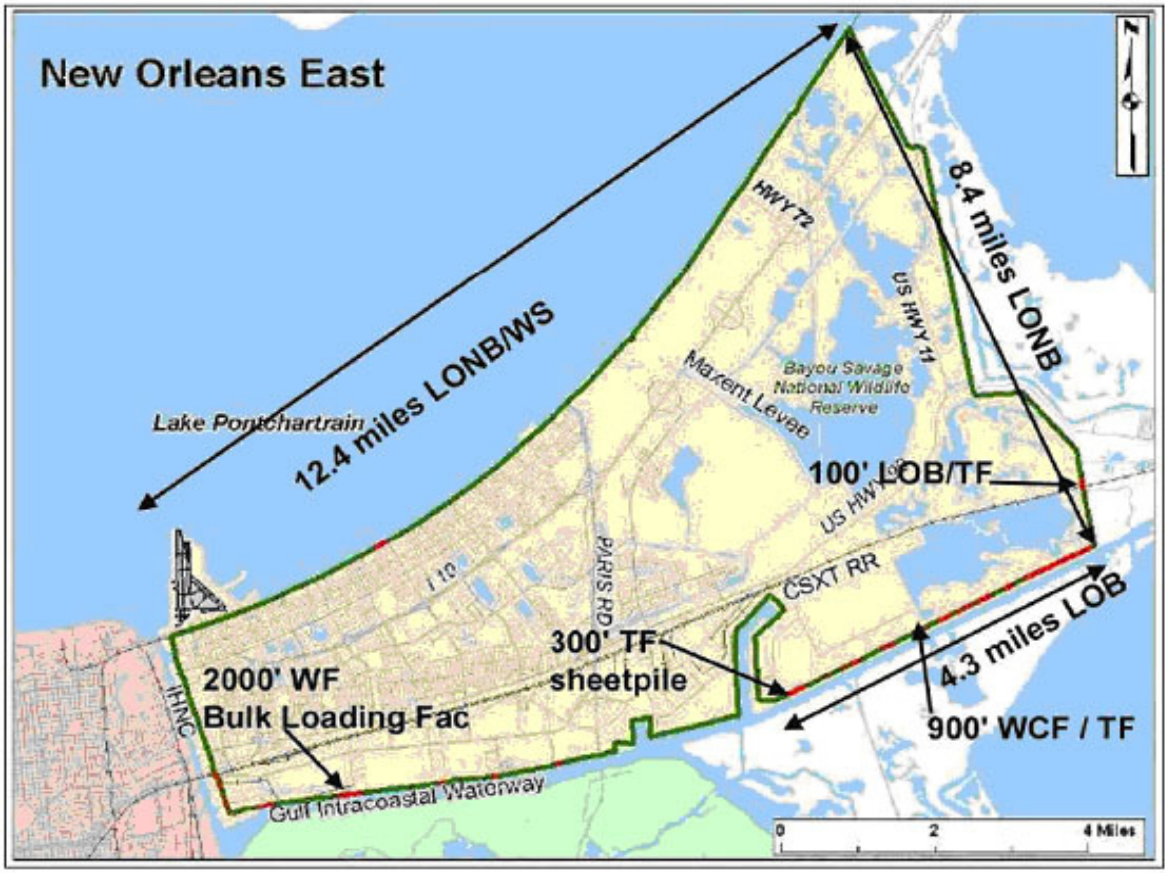


Figure 39. Overtopping and breaching in New Orleans East. IPET developed the terminology shown above to distinguish failure modes, and cause of flooding. See Figure 35 for key (IPET 2006b, V-14-9).

By 0600, an eyewitness at a New Orleans East pump station close to where the MRGO/GIWW enters the IHNC reported that levees on both the north and south banks were submerged. An LSU ADCIRC model hydrograph for the IHNC predicts this (Figure 40). This overtopping event caused minor degradation of the 12 to 15 ft MRGO/GIWW levees on both sides of the funnel throat, and caused about 2,000 ft of I-wall to lean precariously on the north bank (Figure 41). Waves were present and more of a factor than accounted for in the original design (Figure 42), but were traveling down the axis of the channel rather than breaking against the levees, as has been pointed out by IPET (IPET 2006b, IV-250).

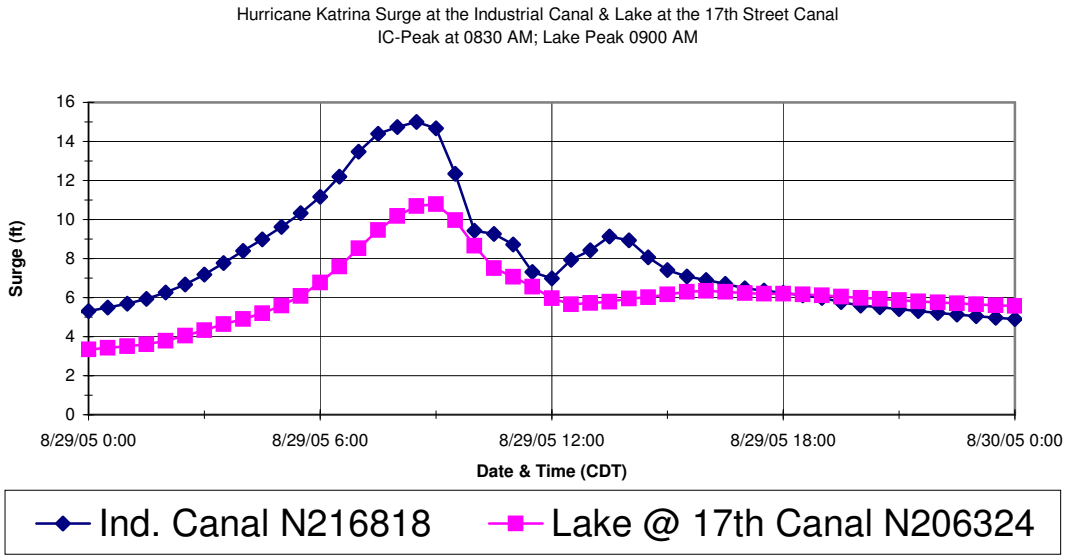


Figure 40. LSU ADCIRC model hydrographs for specific nodes in the IHNC (Industrial Canal) near the Lower 9th Ward and in Lake Pontchartrain at the 17th Street Canal.

Figure 41. View west of leaning I-wall on the north bank of the MRGO/GIWW at New Orleans East port facility. Overtopping caused scour on both sides of I-wall.

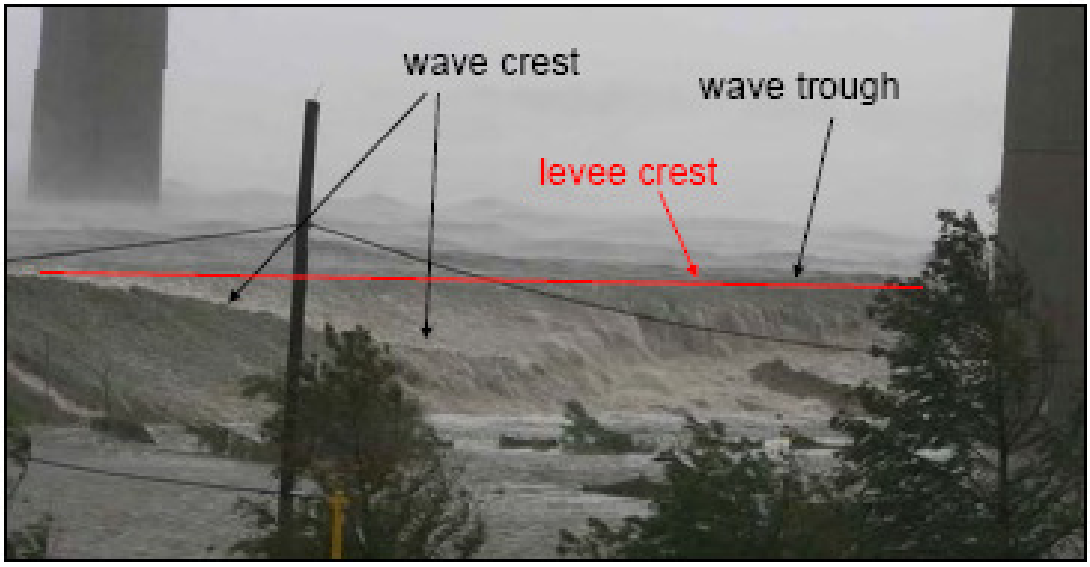


Figure 42. Photo of surge waves propagating up the length of the channel, taken by Entergy personnel showing wave flanks. overtopping levee on north bank of MRGO/GIWW channel looking west toward footings of Paris Road Bridge. Annotation added by IPET (IPET 2006b, IV-250).

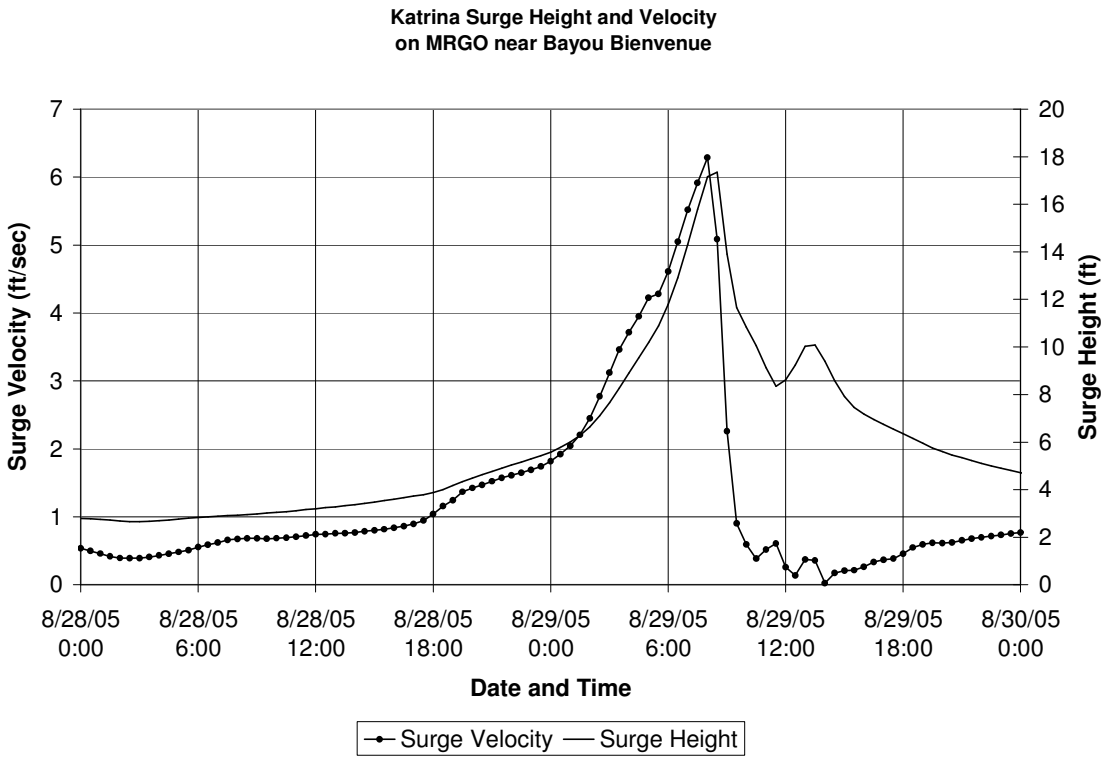


Figure 43. LSU ADCIRC simulation of surge height and velocity in the MRGO adjacent to the Bayou Bienvenue Control Structure.

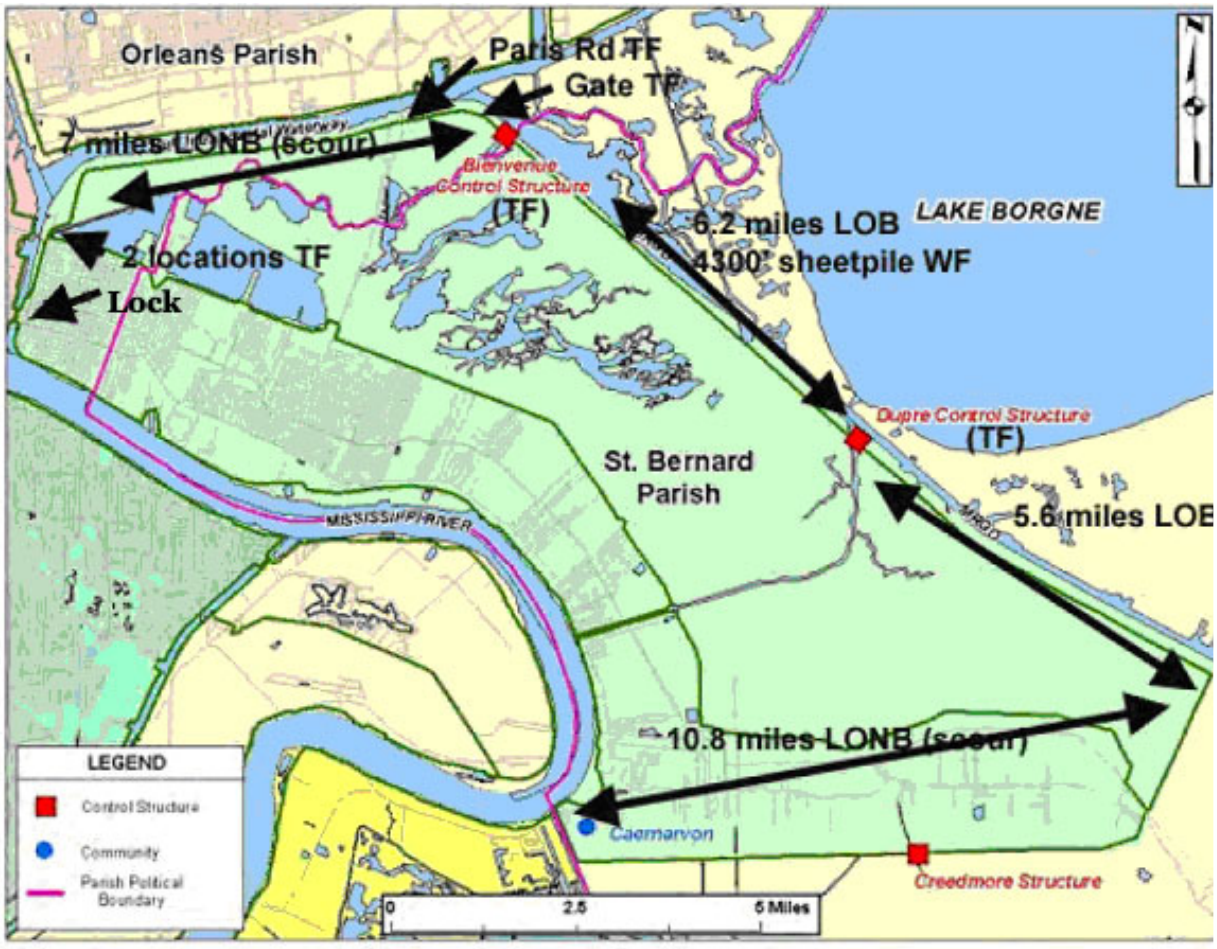


Figure 44. IPET damage assessment on St. Bernard HPS showing 30 miles of levees that experienced some overtopping, and 12 miles of breaches along the MRGO (IPET 2006b, V-14-15). Unmarked north-south channel along the left side of figure is the IHNC.

At around 0500, the surge in the funnel rose to 10 ft and long-period waves from Lake Borgne began breaking on the 12 to 18 ft federal levees fronting the MRGO. This part of the GNO HPS protects St Bernard Parish and the Lower Ninth Ward (Figure 44). The waves initially eroded a wave bench on the unprotected side of the sandy levee face. Discharge from overtopping waves then eroded the back side (Figure 45). The back side scour holes expanded upgradient through the crown, called 'head cutting,' to merge with the wave bench, essentially removing the crown and creating the breach.

This "levee capping" process occurred at dozens of locations, starting at spots that were lower or faced with more easily eroded materials. Eventually, the breaches grew to include virtually the entire 60,000 ft of the levee along the MRGO, most of which was eventually overflowed.

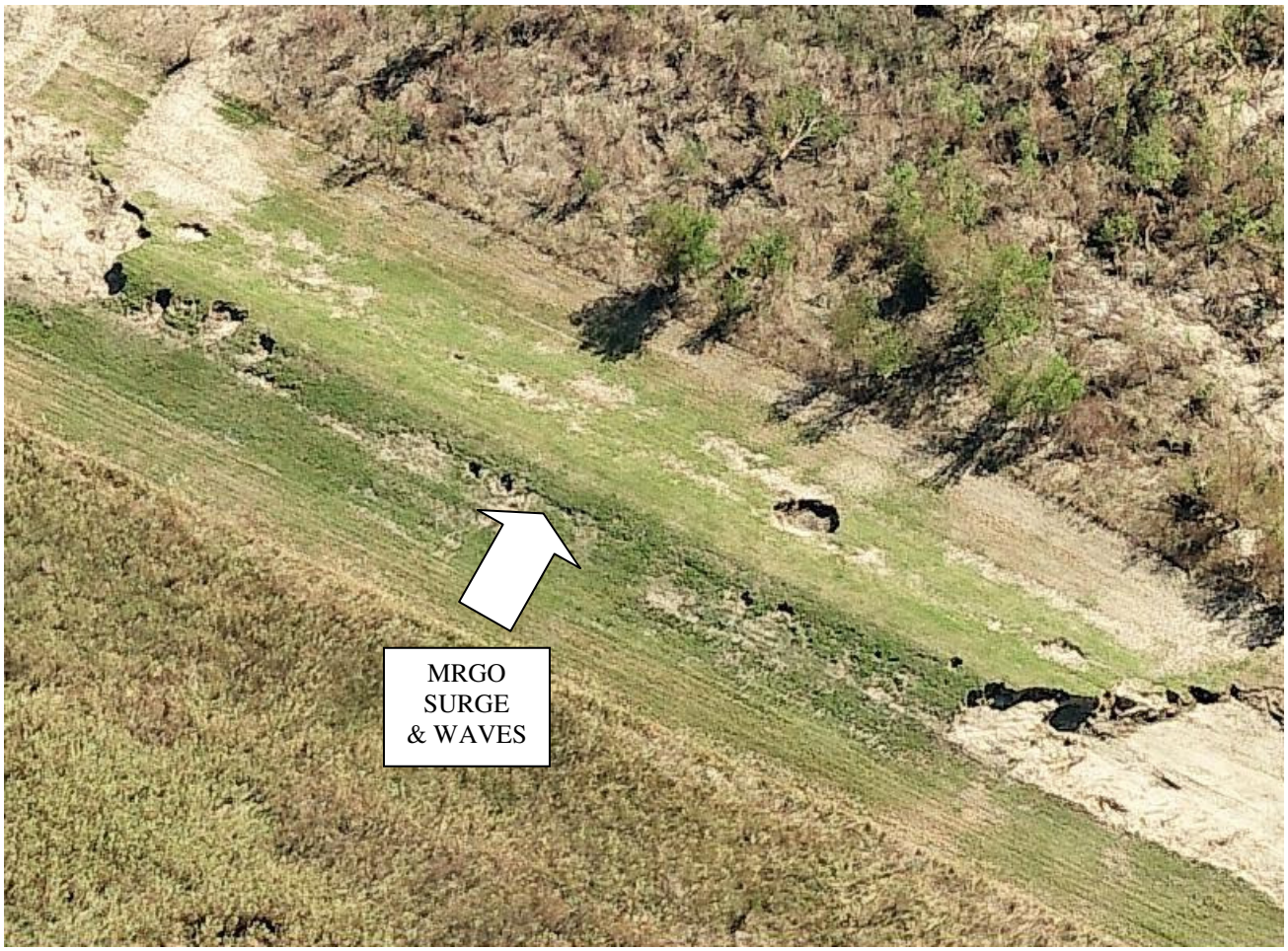


Figure 45. High-resolution oblique Pictometry image of MRGO levee east of Bayou Dupre. MRGO is at bottom, and protected side at top. Photo shows several stages of "levee capping" breaching that initiates with wave bench formation on unprotected side, followed by wave overtopping and back side erosion, which enlarges upstream through the levee crown to join the front side bench.

Breaches formed in this way, as well as erosion of the loosely filled closure plugs adjacent to the Bayou Bienvenue and Dupre control structures (Figure 46), allowed discharge into the wetland reservoir between the MRGO levee and the north bank of the 40 Arpent Canal. This 32,000 acre reservoir between the two levees has a 9 billion cubic foot capacity but filled rapidly so that by 0830 the 7 to 9 ft back levee overtopped. The surge then roared south into St Bernard, flooding everything east and west of Paris Road and eventually draining through the breaches in the Lower 9th Ward (Figure 47).

Figure 46. Oblique Pictometry image of Bayou Bienvenue Control Structure from the southeast, with MRGO to top of frame. East wing wall formed plug for the original bayou and was filled with sand and shell washed out and formed scour hole on protected side (left) prior to overtopping and locally lowered the surge level, as can be seen by the lack of scour trench behind the immediately adjacent surviving sheetpile wall.

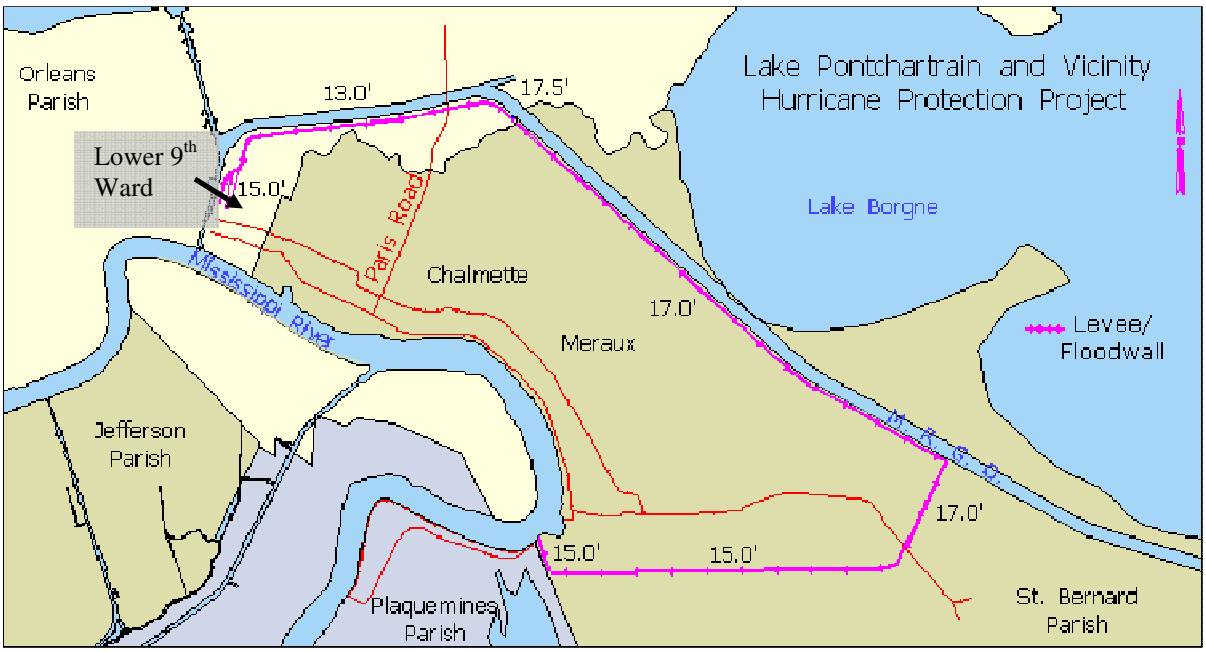
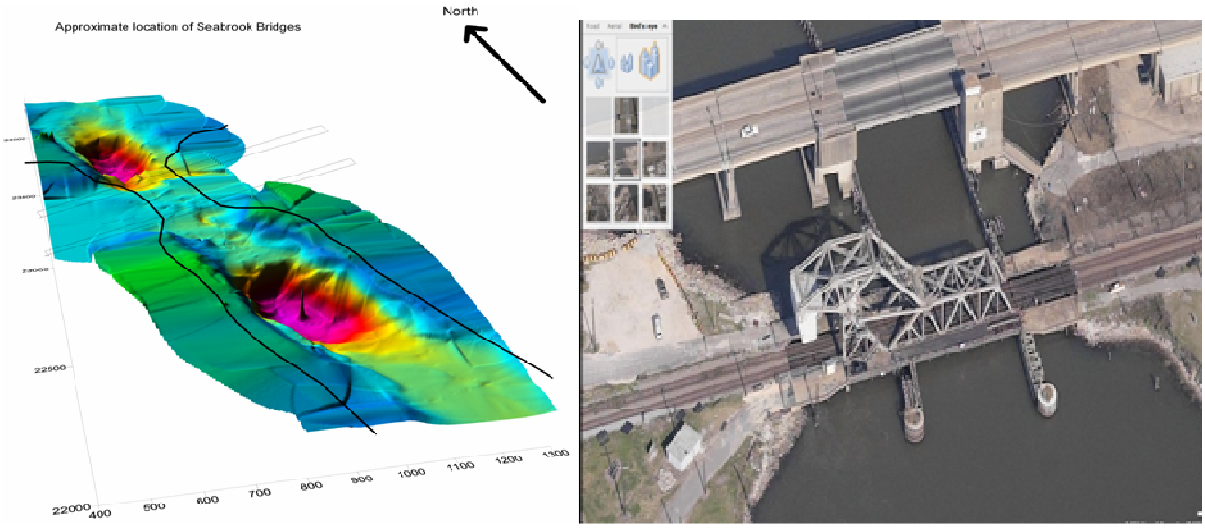


Figure 47. St. Bernard HPS showing federal levees in purple with design level of protection relative to an NGVD29 datum of unknown vintage.

Later Flooding of the Lower 9th Ward and New Orleans East

As the eye of the storm approached the latitude of New Orleans, the surge continued to rise in the funnel (Figures 40 and 43), and this increase in water level was conveyed with little reduction through the MRGO/GIWW connection into the IHNC. The surge jet moving west was directed to the south by a turn in the channel at the junction with the IHNC (Figure 44). There it pushed against the floodwalls and levees protecting the west bank but was ultimately stopped on the south by the lock structure that connects the Industrial Canal to the Mississippi River. Water that could not exit in this direction flowed north toward Lake Pontchartrain, which at 0700 was about 6 ft lower (Figure 40). One indication of the energy conveyed by this flow is the scour that occurred in the IHNC near the entrance to Lake Pontchartrain where two scour holes were eroded to a depth of nearly 90 ft while the normal depth is less than 40 ft (Figure 48).

At approximately 0600 levees and floodwalls along both banks of the IHNC were overtopped, beginning on the south end where the surge was highest. Surviving floodwall crown elevations along the IHNC were surveyed after the storm by IPET and found to range from 11.1 to 13.0 ft (LMSL) (Figure 49). Water pouring over I-wall structures (Figure 40) began to scour trenches on the back sides where the supporting earth levees were unprotected against scour (Figure 50). In the Lower 9th Ward, the influx of water overtopping the levees added to the flooding coming from the GIWW. At the Louisiana National Guard's Jackson Barracks in Arabi, situated on the high ground of the natural levees of the Mississippi River and about 2 miles from the IHNC, flood waters were first seen about 0700, but started rising rapidly after 0730.



Figures 48a,b. Post-Katrina bathymetry acquired by LSU Coastal Studies Institute in the IHNC at Seabrook where it meets Lake Pontchartrain on the north end. The purple shaded zones scour holes on either side of the bridge contraction are more than 80 ft deep in a channel that is otherwise less than 40 ft.

The New Orleans Police Department began receiving increasingly panicky calls from the Lower 9th Ward at 0755. These were relayed to Bob Turner, Executive Director of the Lake Borgne Levee District. He called the National Weather Service in Slidell at 0810 and disseminated this information via their webpage at 0817. There is consensus that the 1,000 ft IHNC break occurred at 0730 when the gage at the USACE lock was reading about 12.5 ft (Figure 40). At approximately the same time, or a little later the northern breach occurred in the Lower Ninth Ward (Figure 37). The nature of the residential flooding changed considerably at this point, increasing in velocity and rising rapidly so that residents had to rush to get into their attics. This timing corresponds well to observations of National Guard officers who saw rapidly rising water after 0730 from their post at Jackson Barracks. Conditions did not improve significantly until about 1400 when the Guard was able to cut their floating boats from the trailers and begin making rescues from roofs (Figure 51).

Some of the engineers who have examined the south breach into the Lower 9th Ward believe it was caused by overtopping induced levee erosion, while others believe that piping and lateral movement may have been a contributing factor. Early reports attributed it to many other factors, including a barge collision. We found that while the barge had indeed struck the I-wall at the south end of the breach, the angle of the steel reinforcing bars in the concrete cap exposed by the collision suggested that the section of wall struck was already leaning having failed earlier. The barge had clipped the end of the already formed breach as it was sucked through. The levee was entirely removed in the breach area and the still-interlocked concrete capped sheetpile of the I-wall that failed was fanned out on the protected side in an intact arc (Figure 52). IPET analysis of the failure indicated that the wall at this location could have withstood overtopping if back side erosion had not removed the levee that supported it (IPET 2006b). ILIT (2006) and Team Louisiana believe under-seepage could also have been a factor. It would appear the main cause of failure at the northern breach into the Lower Ninth is both under-seepage, and loss of levee stability or overtopping and removal of soil support. No doubt both failure mechanisms were at play and both led to the failure.

At about the same time, the breaches on the west side of the IHNC that were initiated earlier enlarged as the water continued to rise. While the integrity of the Orleans Metro HPS was now seriously compromised, none of the breaks that had occurred along the IHNC and GIWW scoured below an elevation of five to seven feet above sea level, leaving resilient "sills." The existence of elevated sills in these breaches meant that they stopped flowing when the canal level dropped below this point (Table 3a).

Residents on the north side of the Chef Menteur ridge in the Little Woods area of New Orleans East began seeing water pouring over the railroad embankment from the south at about 0800 (Figure 38). Water overtopping from the MRGO/GIWW connection had filled the somewhat higher intervening industrial area to the south. Those closer to the IHNC also saw water overtopping floodwalls, but this flooding was limited and did not result in significant breaches.

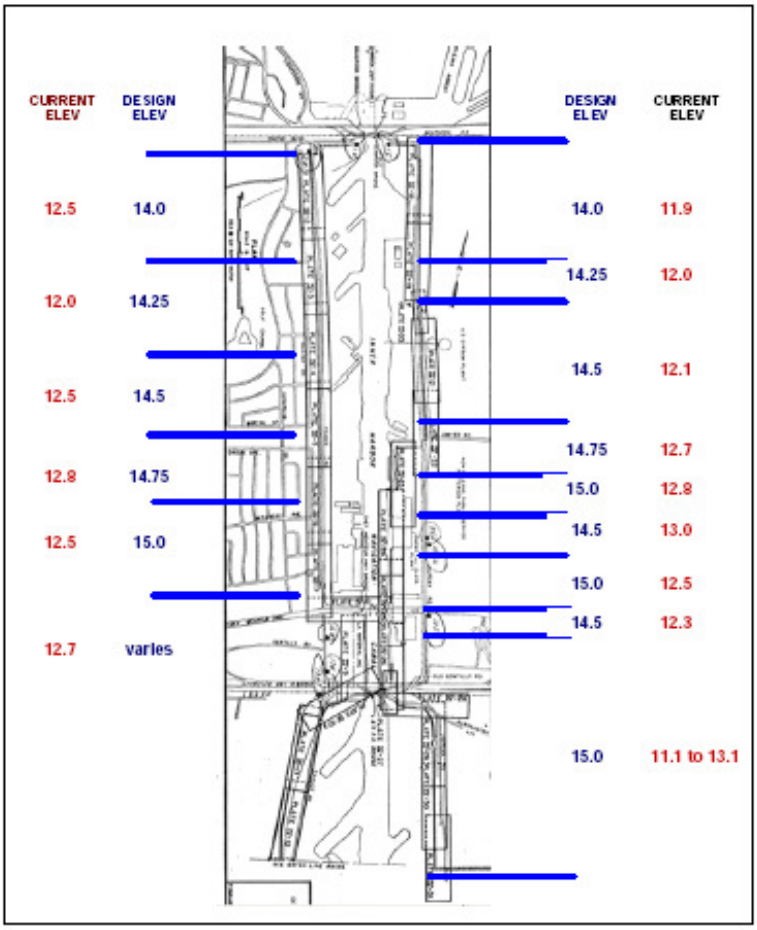


Figure 53. Design versus Current Elevations in IHNC. Design elevations in feet referenced to MSL. Current elevations in feet referenced to LMSL (2001-2005). Grade sectors and current elevations are approximate

Figure 49. IPET post-Katrina survey of floodwall elevations along northern reach of IHNC (IPET 2006b, II-113). Design and post-Katrina elevations are relative to local mean sea level (LMSL05).

Figure 50. Scour trench behind surviving I-wall in Lower 9th Ward.

Figure 50b. View looking south of the north breach into the Lower Ninth ward. Note on the left how the wall is 'standing on its head' and at the south edge of the breach, right side of photo, we see evidence that the wall slid backwards.

Figure 51 View west looking at southern breach (of two) in the Industrial Canal. Lower Ninth Ward in foreground. Orange colored barge once thought to have played a role in the breach can be seen just inside the south end of the breach.

Figure 52. Exhumed I-wall that failed in the south breach into the Lower 9th Ward fanned out on ground behind its original position, with sheetpiles still interlocked and concrete cap to right.

Some overtopping from Lake Pontchartrain was also reported south of the Lakefront Airport where a section of concrete levee wall was almost two feet lower than the adjacent earth levees. Overtopping took place there for about 2 hours (Figure 53). Additionally, a section of lakefront levee near Paris Road was overtopped for about an hour, presumably as a result of waves.

Figure 53. Trench eroded from Lake Pontchartrain water overtopping a floodwall immediately south of Lakefront Airport.

IPET believes the main route of floodwater to New Orleans East was from the MRGO/GIWW funnel throat to the south. But medical personnel located closer to the local levee that follows Paris Road at the Methodist Hospital off Reed Avenue (Figure 38) began to notice water in the streets around 0900. It rose relatively slowly and appeared to come from the west. It did not get high enough to stop their generator until 1030. This was the time recorded on many synchronized electric clocks in the facility. The gradual rise continued and began to enter the building about 1100. It is likely that some of the water that flooded this area came from breaches that occurred along the GIWW levee east of the Michaud Canal slip that filled the marshy area behind the federal levee and then overtopped the local levee (Figure 39).

Breaches along the GIWW affecting the residential areas of New Orleans East were much less extensive than those affecting the MRGO levee on the south side of the funnel. As a result, most flooding was caused by overtopping that stopped once surge levels dropped. IPET reports that the water level in New Orleans East initially got to +1.5 ft in some residential areas close to sources of overtopping, and higher in the industrial land to the south, and in the Bayou Sauvage marsh to the east. Once the surge abated on the GIWW, MRGO and IHNC, the surge water that had entered then spread out, dropping in some areas and rising in others until it reached equilibrium more than a foot below mean sea level. Because Little Woods is so low, however, with many homes between 8 and 12 ft below sea level, even flooding to this modest elevation had catastrophic consequences.

Flooding of Orleans Metro from Lake Pontchartrain

As the eye of the storm started to cross the main Lake Pontchartrain tidal passes, winds over Lake Pontchartrain backed to the north pushing surge water and waves against the levees along the south side of the Lake (Figure 40). The HPS along the lakeshore is pierced by channels in five places between the Jefferson/St. Charles Parish Return Canal west of the Jefferson Parish suburbs, and Chef Menteur Pass to the east of New Orleans East. The westernmost drainage or outfall canal is the 17th Street Canal on the Orleans/Jefferson Parish line, followed by the Orleans Avenue Canal, Bayou St. John, London Avenue Canal and the IHNC. All were open to the Lake during the storm except Bayou St. John which has a floodgate that functioned very effectively.

At 0900, surge levels rose as high as 12 feet near the mouth of the IHNC, and were above 10 ft (NAVD88) to the western side of the Jefferson basin. At the entrance to the 17th Street Canal, restaurants and boat houses built over water at the lake entrance broke up as did camps and homes on the lake shore outside the levee, and the debris ended up in the canal. This debris formed a porous dam against the Metairie-Hammond Highway Bridge as the waves drove the debris against the bridge. This debris dam growth was then enhanced after the breach induced flow into the canal (Figure 54).

We have been surprised by how early – and at what relatively low canal water levels – IPET now believes the I-walls on the London Avenue and 17th Street Canals failed. The surge peaked at 0900 at about 10.5 ft. feet above mean sea level at the mouths of these canals, but IPET has now placed most failures at between 0600 and 0730 when the Lake Pontchartrain surge on the south shore was between 6.5 and 8.2 ft (Table 3a).

Figure 54. Debris dam at Metairie-Hammond Ave Bridge, 17th Street Canal. (Source EPA).

The three I-wall failures on the 17th Street and London Avenue Canals have been subjected to far more scrutiny than the other 47 GNO HPS breaches identified by IPET. They have been termed "structural" failures, like the north breach into the Lower 9th Ward, because the levee foundation gave way at canal water levels that were within the safe operating range specified in the design. LSU researchers were the first to note the lack of overtopping associated with these failures, and this was an impetus for subsequent investigations. We discuss the specifics of each of these failures in more detail in another section, focusing on what the design engineers knew or should have known at the time. Here, we look at the timing.

The difference between the IPET position supporting a very early initiation of the drainage canal breaches, and what most eyewitnesses reported, may be partially semantic. IPET is seeking to document the time of first movement of the levee foundation while most observers noticed the breach later, when it was fully developed. IPET has reported, for example, that panels of the I-wall on the east side of the 17th Street Canal began leaning as early as 0630, but most eyewitnesses agree that little water escaped until the main failure at between 0830 and 0915. I-wall panels on other levee reaches also moved during the storm, for example the I-wall protecting the

western side of Jefferson Parish (Duncan Canal), but this did not provoke comment at the time because the incipient failure did not lead to a breach (Figure 55).

Figure 55a. I-wall movement and partial failure on Jefferson/St. Charles Parish Return Levee (Duncan Canal) that was not noted until well after the storm.

Figure 55b. Times-Picayune

IPET has reported that both breaches on the London Avenue Canal were initiated before 0800. IPET gives the sequence of drainage canal breaches as starting with the failure to the east out of the 17th Street Canal at Robert E. Lee Boulevard at between 0600 and 0630, with water in the canal at 6.5 ft, followed by the Mirabeau (south) breach on the east side of the London Canal at between 0630 and 0700 when the canal was at 7.1 ft (Table 3a). IPET believes the Robert E. Lee breach on the west side of the London Avenue Canal near the north end of the I-wall occurred between 0700 and 0730 when the canal was at 8.2 ft. It is difficult to tell how much of this timeline is based on actual observation and how much is derived from failure models.

17th Street Canal Breach

By 0930, the eastern levee of the 17th Street Canal had broken and the breach had reached full development a hundred yards inland from the hurricane proof bridge crossing the canal near the Lake shore (Old Hammond Highway). Surge waters immediately started to flow into the western portion of Orleans Parish (Figure 56). None of the residents to whom we spoke recalls seeing the 17th Street Canal breach at the moment it occurred. The Chief of the New Orleans Fire Department received reports from personnel who were in a high-rise near the mouth of the canal. They

estimated that the time was 0927. A resident who was reporting by cell phone on conditions four blocks from the breach made four calls between 0930 and 1000. He did not see water in the street until just after 0937.

Figure 56 Aerial view of 17th Street Canal Breach looking east, showing the separation of the I-wall from its supporting levee and the lateral displacement of the levee. Water is flowing from the Lake through the breach a day later (Source EPA).

Eye witnesses reported the flood waters rose rapidly. As these waters gushed up the canal from the Lake, the debris floating in the canal mouth was swept up against the Metairie-Hammond Highway hurricane bridge, which restricts the cross section of the canal due to its piers and low deck level (Figure 54). The debris packed up as a substantial dam that restricted inflow, and served as a very effective barrier to waves that might otherwise have propagated up the canal from the lake. The ILIT have reported that wall on the west bank of the canal opposite the break, the Jefferson Parish side, was also close to failure and experienced some deflection (ILIT 2006).

The London Avenue Canal

Surge levels at the entrance to the London Ave Canal were essentially the same as those to the east at the mouths of the 17th Street and Orleans Avenue Canals, and Bayou St. John, peaking close to 11 ft (NAVD88) around 0900. The entrance of the Orleans and

London Avenue Canals differs from that at 17th Street in that the I-walls begin farther inland, behind an S-shaped entranceway designed to absorb wave energy (Figure 57).

Figure 57. View from Lake Pontchartrain of meandering entrance to London Avenue Canal on left and straight entrance at 17th Street Canal with bridge for wave barrier.

Our information agrees with that of IPET in placing the Mirabeau (southern) break as the first of the two London Avenue Canal breaches. We have a reliable clock and eyewitness video data from homes adjacent to the breach location that shows it failed before 0920. Flood waters immediately poured into the City, displacing one home off its foundations and sliding it 90 feet across a road (Figure 58). The video was taken just over an hour later at 1030. By this time the video shows water up to the eaves of houses closest to the breach and a water depth of about 5 ft two blocks from the breach. Flood water from the Lake streamed down the Canal towards this breach. While water levels were lowered close to the breach, elsewhere the surge level remained high, precipitating the second failure to the west into the Filmore neighborhood just south of the Robert E. Lee Avenue Bridge (Figure 59).

It is possible that this breach did not reach full development until after the peak of the surge, as the canal was falling. All investigators studied the floodwall on the east bank opposite of the Robert E. Lee breach, which appears to have been arrested in a state of incipient failure. The consensus of ILIT, IPET and Team Louisiana is that the west bank

breach relieved pressure on the east bank wall. But it is also possible that the arrest could have occurred as the Lake level was falling after the peak (Figure 60).

Figure 58a. Looking south towards Mirabeau break on London Avenue. Slab brick house was pushed off its pilings across the street from right to left. Sand dunes deposited by floodwaters in foreground.

Figure 58b. View looking north of London Ave Mirabeau breach. Note deep scour hole and that wall segments appear to have fallen into hole

Figure 59. Looking south toward Robert E. Lee (north) break on London Avenue.

Figure 60. Looking south at leaning I-wall on east side of the London Avenue Canal opposite the Robert E. Lee breach (north).

The Orleans Avenue Canal

The Orleans Avenue Canal lies between the 17th Street and London Avenue Canals, and experienced the same surge sequence. The concrete capped levees of the Orleans Canal held, but the capping project had not yet been completed. Earthen levees that were awaiting floodwall installation lined the last 400 ft of the southern terminus of the canal, adjacent to the pump station. These levees, about 9.0 ft high, compared to the 13 to 14 ft crown of the I-walls, were overtopped for at least two hours (Figure 61 a & b). This was one of the unfinished elements of the GNO HPS. This overtopping caused significant flooding on the west side of City Park, but it also lowered the pressure on the floodwalls between the pumping station and the lake to some degree. In effect, this unfinished portion of the Orleans Canal protective structure functioned as a spillway for two hours, lowering the level of the canal, at least near its southern terminus. (van Heerden and Bryan 2006)

Water poured unabated through the three breaches of the drainage canals in the Orleans Metro for more than 50 hours, until the water inside the "protected" area equalized with that in the lake at about 3 feet above sea level, and with the "average" home in the flooded sections standing in six to nine feet of water. On the east side of the IHNC, in contrast, the influx of floodwaters occurred for less than six hours. This important difference developed because the canal floodwall breaches scoured down to below sea level, to 20 ft or deeper for the Mirabeau breach on the London Avenue Canal, while those that compromised the GNO HPS elsewhere did not erode below + 5 ft (Table 3a, b).

Figure 61a. South end of Orleans Avenue Canal I-wall on east side, showing uncapped earth levee that was overtopped, and scour around the utility pole.

Figure 61b. View missing capping on east side of canal, looking north. Capping also missing from west bank.

Late Stage Flooding of St. Bernard

While Orleans Metro was flooding from Lake Pontchartrain through the drainage canal breaches, the stage was set for a second disastrous inundation of St. Bernard east of Paris Road. This area had already received some flooding from the west caused by the breaches and overtopping in the Lower 9th Ward, and from the south across the crown of the Caernarvon to Verret levee along the south side of the HPS (Figure 44). The Caernarvon to Verret levee was overtopped briefly for about 1.5 miles where it was less than 13 ft high, and this happened when the water was highest at that point, at about 0800. This overtopping was relatively brief, and the levee was protected from waves by a vast wetland area to the south. Despite overtopping, the levee crown experienced only minor damage, perhaps also due to a construction design that included a substantial clay cap placed over a core of sand pumped from the Mississippi River (Figure 62).

Flooding from overtopping and breaching of the IHNC into the Lower 9th Ward together with the more limited overtopping of the Caernarvon to Verret Levee account for about 10 percent of the total flooding of the protected area within the St. Bernard HPS. At around 0830, the most substantial flooding event in any part of the GNO began as the 40 Arpent back levee, with a crown elevation of between 7 and 9 ft, experienced massive overtopping from the wetland buffer area to the north and east (Figure 44).

Figure 62. Looking west on the Caernarvon to Verret Levee that protects the south side of St. Bernard drained area east of Chalmette. This levee was overtopped for about 1.5 miles where it was between 12.5 to 13.0 ft high, but was protected from waves by an extensive marsh to the left. It suffered little damage.

This was a very destructive overtopping event that substantially destroyed homes in subdivisions in the Meraux area that backed up to the 40 Arpent Canal (Figure 63). The water did not enter uniformly over the whole levee but formed 'rivers' that started at several low spots, joined up in the 40 Arpent Canal that was just inside the levee, and raced preferentially through subdivisions and other cleared areas as the flood flowed toward the Mississippi River levee. This flooding could only have started after the vast wetland impounded between that levee and the MRGO levee, nearly 32,000 acres, had filled to the elevation of the lowest reaches of the 40 Arpent Canal levee (about 7 ft).

The water that filled the impounded St. Bernard wetland and flowed over the 40 Arpent Canal levee into the developed and protected part of St. Bernard Parish came initially through breaches in the MRGO levee. Filling the impounded wetland to a level of 7 ft required introduction of nearly 9 billion ft^3 of water across the MRGO levee within a

Figure 63. View northwest toward 40 Arpent Canal and Levee in Meraux showing homes destroyed down to the slab by flooding from the MRGO across the back levee. The levee crown is visible in background.

relatively narrow time window. That time window is available from the LSU ADCIRC hindcast of surge on the MRGO (Figure 43) that has been corroborated by gage data from the IHNC lock (Figure 16) and by the HWM analysis (Table 2).

The surge window on the MRGO can be defined by the time the water level exceeded a specified elevation. If that elevation is defined as 6 ft, then the surge lasted 18 hours, but if it is defined by the time above 10 ft, which is about the height any previous surge had attained on the MRGO levee, the period is reduced to 6 hours, starting at about 0500. Because the problem is to explain water levels in excess of 10 ft within the St. Bernard protected area, we are particularly interested in the exceedence interval for this value. Water levels in MRGO and the 10 ft exceedence interval constrain the St. Bernard flooding event, as does the maximum water elevation attained inside the protected communities, about 11 ft (NAVD88). If too much water enters, the interior water level would have gotten higher than what was observed, while too little inflow does not fill the protected area to the observed level. Finally, we know that water did not enter the wetland buffer area uniformly along the entire 12 mile length of the MRGO levee. This

levee, like the 40 Arpent Canal levee, was not initially of uniform height but had a crown that ranged from 12 to 18 ft (NAVD88). The maximum surge elevations without the wave component along this levee ranged from 16 to 18 ft, and so were comparable in most places to the pre-Katrina crown elevation (Figure 64). Post-storm surveys disclosed that about half of the MRGO levee had experienced significant degradation or breaching. If the levee had not failed, it should have experienced general overtopping only for about an hour prior to the surge peak at 0830, and this window would not have provided enough water to explain the filling of the St. Bernard protected area.

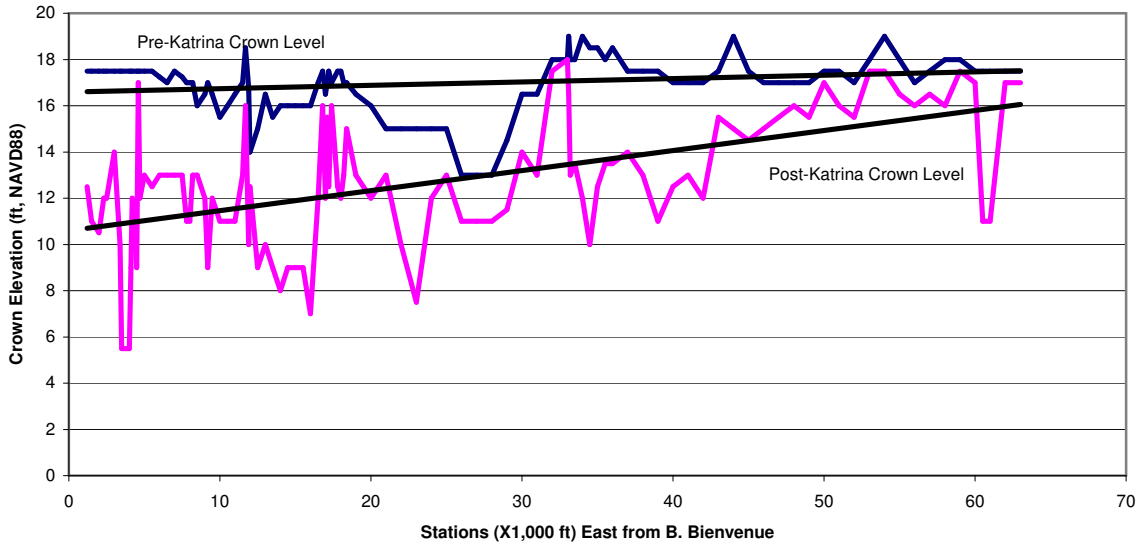


Figure 64. MRGO levee degradation from near junction with GIWW to Verret (derived from IPET 2006b, IV-16-2 to IV-16-10). Subtract 0.6 ft for MSL.

A simple flooding model was adapted from IPET to satisfy the constraints discussed, while delivering the requisite water volume across the MRGO levee in the 6 hours available (IPET 2006b, IV-240). We assume for this model that wave overtopping of the lowest areas begins when the surge attains 10 ft. This is the level at which IPET estimates that low-frequency waves began breaking on the levee face and would have initially sent water over the crown in the lowest sections. This surge level occurred on the southern leg of the funnel at about 0500, at least an hour before landfall (Figure 44).

As the surge rose, the zone of wave-induced overtopping and levee degradation expanded until the peak was attained at 0830. It was assumed that the levee crown relatively quickly attained its post-Katrina condition once overtopping by waves began for any given reach (Figure 64). Flow over the levee was modeled using the IPET weir flow equations, first for wave overtopping and then for overtopping by waves and surge. Outflow from the inundated areas, both wetlands and developed zones, begins

once the MRGO drops below 10 ft at 1030. We also included overtopping from MRGO Reach 1, the 7 mile reach that connects the funnel with the IHNC.

Our results for overtopping flow were similar to those calculated by IPET, with the addition of flow from Reach 1 (Figure 65). The rate of transfer of water across the MRGO alignment peaks at around 0830 at more than 2.0 million cubic feet per second (cfs). This is an extraordinary volume comparable to a large flood on the Lower Mississippi River. But the volume of the 32,000 acre wetland buffer that must be filled before the 40 Arpent Canal levee is overtopped is also very large and the 40 ft deep MRGO channel provides an efficient conduit for supply of this water. This model predicts that that filling of the buffer will take nearly 2.5 hours, delaying the onset of overtopping of the back levee until 0830 (Figure 66).

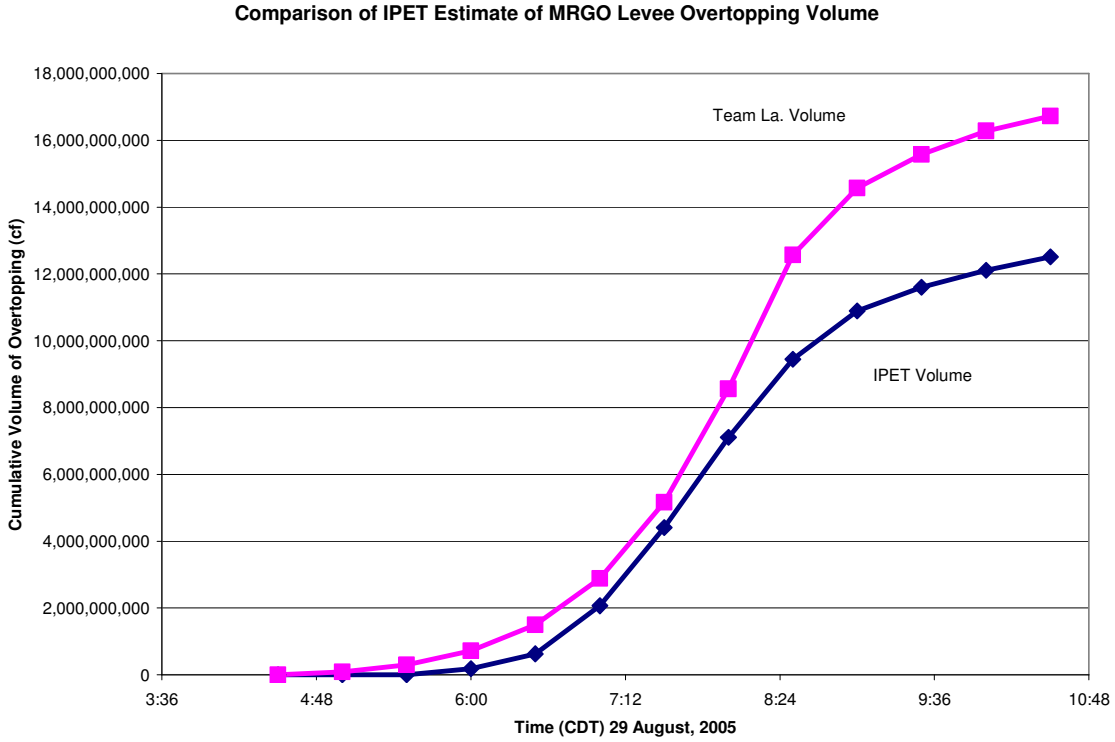


Figure 65. IPET and Team Louisiana estimates of cumulative water volume introduced by waves and surge across the degrading MRGO levee crown. Team Louisiana includes Reach 1 of MRGO.

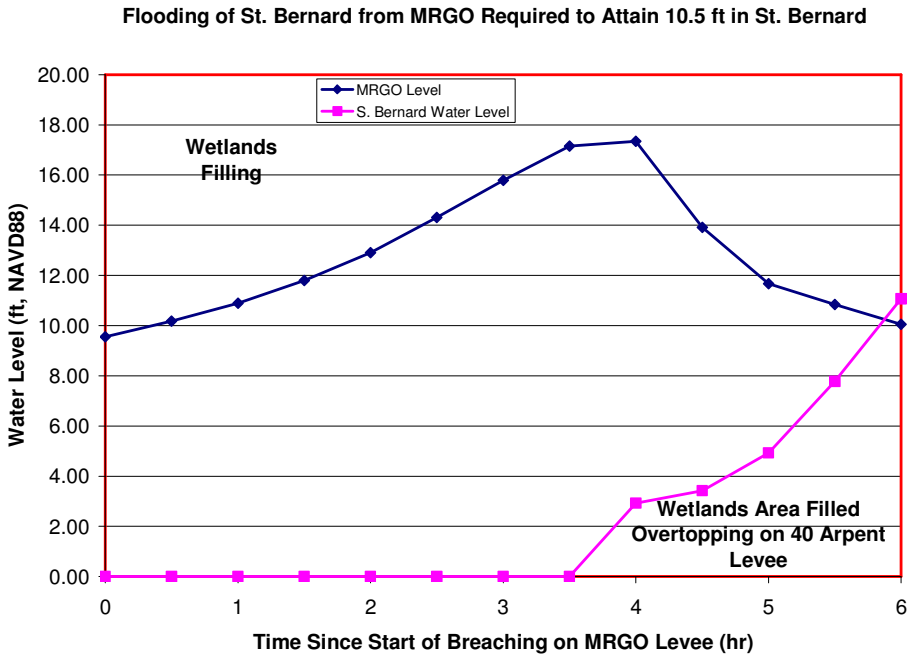


Figure 66. Water level variations as surge Sr. Bernard wetlands and then the protected area of St. Bernard parish.

If the onset of breaching were delayed until the MRGO surge elevation reached 12 ft, then the spreading of levee degradation and failure would have to advance more quickly to attain the required length, and make up the necessary discharge. Under no realistic scenario, however, is the filling of the buffer wetlands an instantaneous event, but almost certainly required hours. Similarly, given the degradation mechanisms available – particularly the levee "capping" described in the previous section - it is unrealistic to assume that breaching of many miles of the MRGO levee occurred at exactly the same time.

Rates of Flooding

The GNO is composed of many sub-basins and neighborhoods that experienced different time-histories of flooding (Figure 67)
(See also www.hurricane.lsu.edu/floodprediction). Four major breaches, and some minor ones, introduced water into the Orleans Metro, along with overflow in the uncompleted floodwall section at the south end of Orleans Canal, and 3.2 miles of floodwall and levee on the west bank of the IHNC (Table 3a, Figure 35).

IPET constructed a series of hydrographs for flooded sub-basins based on stopped clock, video and time stamped digital photographs taken by witnesses (Figure 68). If a camera was pointed sequentially out a window at the same object like a light pole, then

various elevations on the pole were later surveyed to determine the time at which they were submerged.

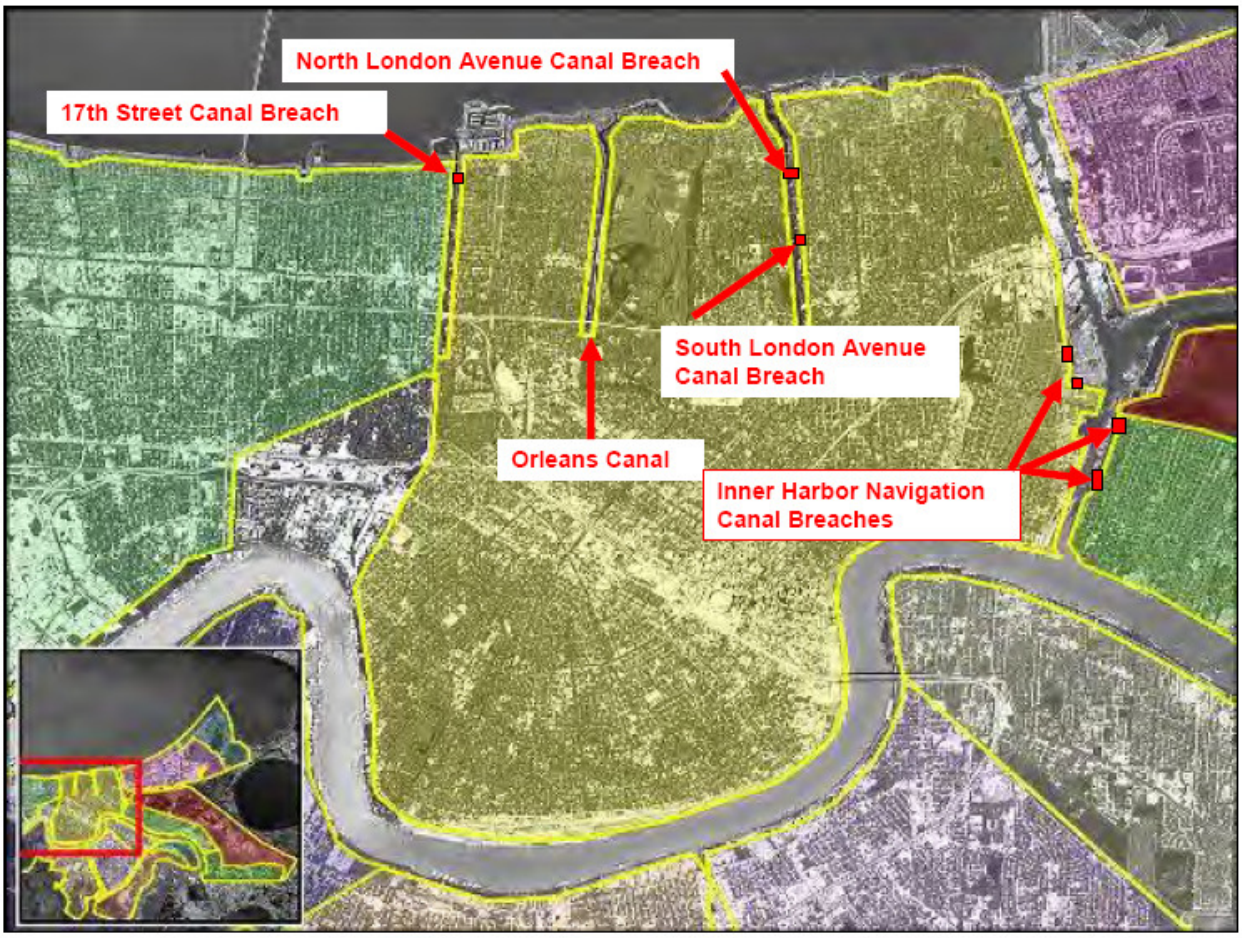


Figure 67. Structural Breaches of the HPS in the Orleans Metro and Lower 9th Ward Districts (IPET 2006b, V-7)

The first neighborhood to flood in the GNO, and one of the fastest rising floods observed anywhere was west of the IHNC container port facility where a significant breach opened early into the Desire and Bywater districts (Figure 35). This is a small sub-basin between the Mississippi River and Gentilly ridges. The maximum rate of rise there was approximately 6 ft/hr, commencing about 6:30, but the rise lasted for only 3 hours, after which the level dropped (Figure 68). Water quickly overtopped the Gentilly ridge and flowed north into the Gentilly neighborhood, but also west into the Palmetto Canal which serves the New Orleans Mid-City area. This area and the lower area of Broadview started rising at this time, but filled very slowly afterward (Figure 68), taking 56 hours to reach the equilibrium level of approximately 2.8 ft NAVD88 (Table 4).

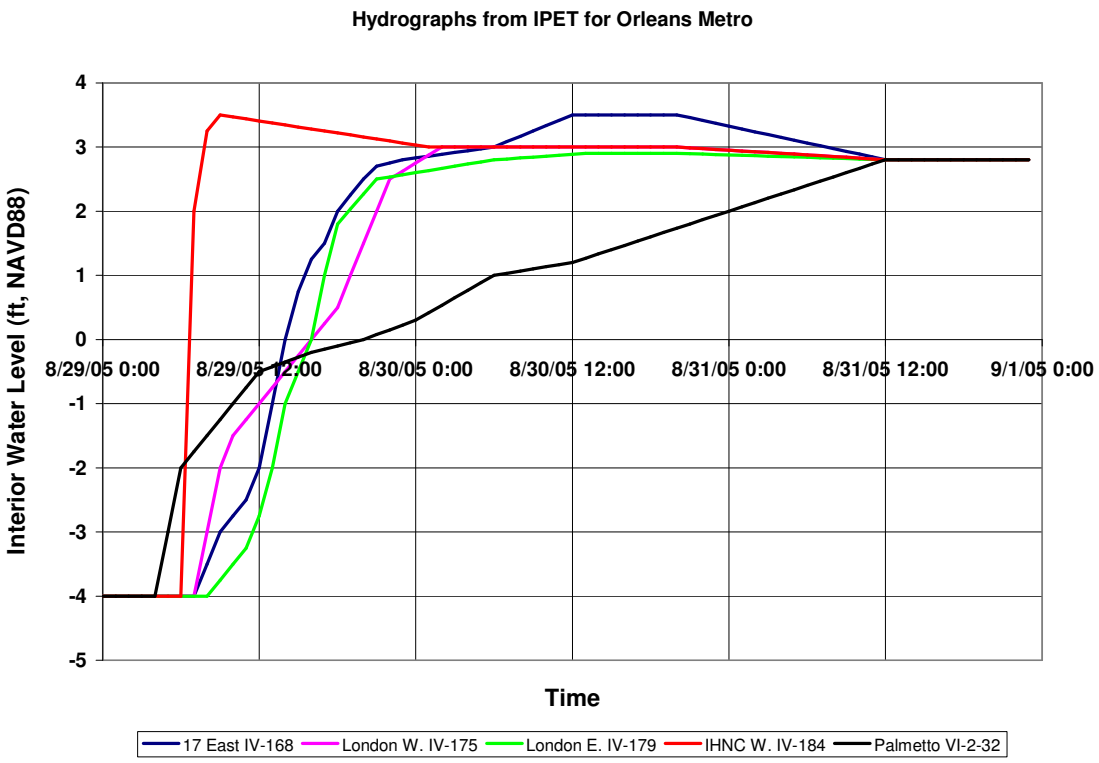


Figure 68. Flood hydrographs for different districts of the Orleans Metro Area adjacent to breach sites. The slower rising Palmetto Canal record documents conditions in the part of the Central Business District that flooded. IPET (2006b) page references noted in legend.

The drainage canal breaches on the 17th St. and London Avenue canals inundated the Lakeview area north of the Gentilly/Metairie ridge and south of the higher reclaimed land along the Lake Pontchartrain shoreline. The flooding histories in the sub-basins that divide up the lowest part of Orleans Metro (Figure 68) are similar but were influenced by subtle terrain features that allowed water to pond before cascading into another area (Figure 32). The basin between the London Avenue Canal and the Bayou St. John natural levee to the west is smaller than those fed by the other two breaches. It filled in 19 hours compared to 29 for the area between the 17th St. and Orleans Ave. Canals, and east of the Mirabeau Avenue breach on the London Avenue Canal (Table 4). (See also www.hurricane.lsu.edu/floodprediction/)

Water coming from the west bank London Avenue Canal breach into the Filmore neighborhood reached wall mounted clocks in the vicinity of the breach within 1 to 4 hours (Figure 32). Water levels there rose most quickly immediately after the breach (Table 4). The opposite is true for the east bank London Avenue breach at Mirabeau which introduced water into a vast area extending to the IHNC. There, water took 11 to

13 hours to reach the wall-mounted clocks, and the most rapid rate of rise was experienced shortly after noon (Table 4).

One surprise in the IPET hydrograph analysis was how quickly and early flooding occurred in the residential area of New Orleans East to the west of Paris Road (Figure 38). IPET depicts the flooding as taking place primarily as a result of water flowing over the low levees and floodwalls along the MRGO/GIWW, in the reach that constitutes the drain of the funnel. This overtopping flow crossed relatively high, built-up industrial land, an elevated railroad bed and the Chef Menteur ridge to reach the lower, Little Woods neighborhood that extends to the lakefront (Figure 29). It is apparent, however, that when this water arrived, it came in great quantity, causing maximum rates of rise of 5 to 6 ft/hr (Table 4), though the initial event was over within 3 hours (Figure 69). The two IPET hydrographs from this area suggest that the water moved almost as a wave from south to north, with a crest elevation at about 1.5 ft. The water level quickly dropped almost 3 ft from this high as the flood spread, reaching equilibrium well below sea level.

Flooding throughout St. Bernard Parish and the Lower 9th Ward was characterized by higher flow velocities and more rapidly rising waters than observed elsewhere in the GNO. Observers in the St. Bernard basin reported flows with standing waves moving down streets. Houses were destroyed or moved tens of blocks. This occurred elsewhere in the GNO only within a few hundred feet of major breaches, but was widespread in the St. Bernard Basin. Water reached the greatest elevation and depth within the GNO in the Lower 9th Ward and St. Bernard (Figure 70). The Lower 9th experienced a rate of rise of 8 ft per hour, and perhaps faster, in the area adjacent to the IHNC breaches, beginning after 0700 (Table 4).

It appears that the maximum elevation reached tended to increase slightly to the east in St. Bernard, so that the high water at Chalmette was about 0.5 ft higher than in the Lower 9th. The prevalent pathway for floodwaters after about 0830 was from the MRGO, across the submerged St. Bernard wetlands, and then over the 40 Arpent Levee. Once this surge finished filling the wetland storage had come came over the 40 Arpent Canal levee, the IHNC breaches that began the flooding became important drainage points as the IHNC water level dropped.

Maximum rate of rise in areas affected by the drainage canal breaches and downtown was 1 ft/hr, but the mean rate of rise ranged from 0.12 to 0.37 ft/hr in downtown and west of London Ave. Canal, respectively (Table 4).

Table 4. Katrina Flooding from IPET Hydrographs for Regions of the Greater New Orleans East Bank

Regions with IPET (2006b) Volume/Page Reference

Parameter	17th St. East IV-168	London West IV-175	London East IV-179	IHNC West IV-184	Downtown Palmetto VI-2-32	East Orleans South IV-191	East Orleans North IV-191	Lower 9th West IV-194	Lower 9th East IV-194	Chalmette IV-194
Time Max Rise 29 Aug. 05 (CDT)	12:30	8:30	13:30	6:30	6:30	8:30	9:00	7:00	8:00	8:30
Maximum Rise Rate (ft/hr)	1.0	1.0	1.0	6.0	1.0	5.5	6.0	8.0	6.0	6.0
Rise Period (hr)	29	19	29	3	56	3	2	9	9	6
Mean Rise Rate (ft/hr)	0.26	0.37	0.24	2.50	0.12	3.75	3.75	1.81	1.83	2.83

Near the west bank breaches on the IHNC, however, the maximum rate of rise was 6 ft/hr, with a mean rate of 2.5 ft/hr. So the flooding occurred more quickly there.

Surprisingly, the rate of rise in New Orleans East was even more rapid than west of the IHNC, with maximum rates of 5.5 and 6.0 ft/hr in the eastern part of the developed area. The mean rate of rise for the developed area protected by the St. Bernard HPS was 3.9 ft/hr (Table 4). The most rapid rates of flooding were measured in the Lower 9th Ward, between 6 and 8 ft/hr, but the mean rate was only 1.8 ft/hr. Farther west in Chalmette, and probably all of St. Bernard, the maximum rate of rise was 6 ft/hr, with a mean rate of 2.8 ft/hr (Table 4).

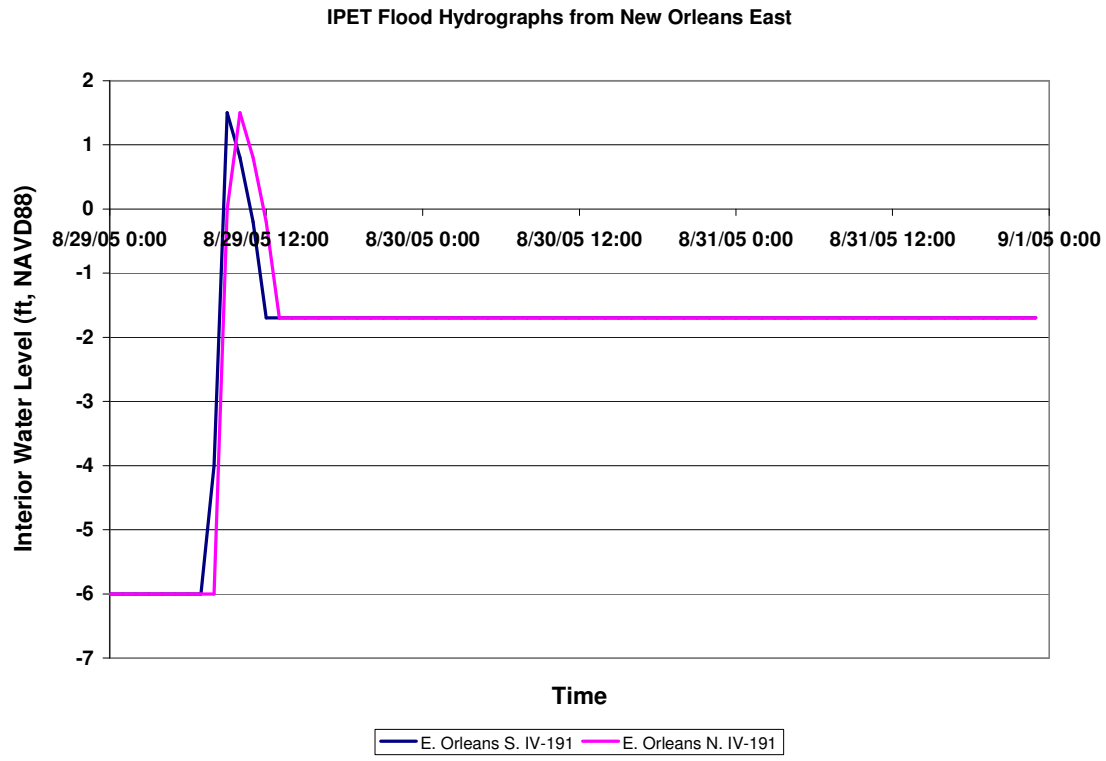


Figure 69. Rates of flooding in Little Woods area of New Orleans East. The south point flooded slightly earlier as it was closest to the MRGO/GIWW overflow source. IPET (2006b) page references noted in legend.

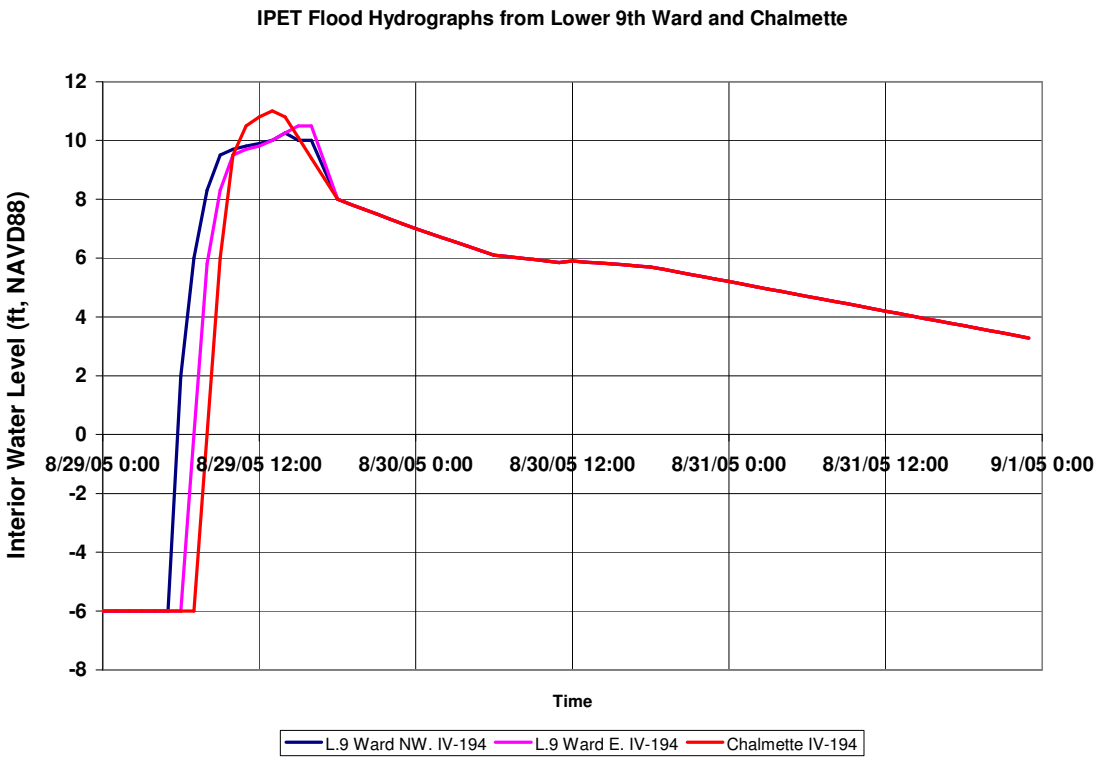


Figure 70. Flooding in the Lower 9th Ward and St. Bernard Parish. IPET (2006b) references given in legend.

Reflooding from Hurricane Rita

Twenty-six days after Hurricane Katrina devastated southeast Louisiana, Hurricane Rita made landfall near the Texas border more than 150 miles to the west and destroyed communities in the coastal southwest. But the GNO also experienced high water once again. Water level rose to more than 5 ft in Lake Borgne and Lake Pontchartrain and overtopped a 6 ft high railroad embankment on the south side of Interstate-10 just west of the GNO (Figure 71). While most of the city had been pumped almost dry by this time, closure of levee breaches was still in an early stage. The Orleans Metro drainage canals that had been conduits for much of the flooding of the City were closed temporarily by sheetpile walls driven into the canal beds on the lake side of the hurricane bridges, and this prevented flooding through them. On the other hand, this could not be done for the IHNC, where repairs had been made with light limestone chips, and several Katrina breaches were reactivated. The most spectacular of these was the large southern breach into the Lower 9th Ward (Figure 72). But flooding through smaller breaches along the west bank of the IHNC also flooded adjacent portions of the Orleans Metro area in the Pontchartrain Park and Gentilly Woods subdivisions (Figure 73). Hurricane Rita flooding pushed back the schedule for dewatering the GNO by nearly two weeks, and exacerbated the difficulty of breach closure efforts on the IHNC.

Figure 71. Hurricane Rita surge in Lake Pontchartrain at 0700 overtopping 6 foot high railroad embankment west of New Orleans in LaBranche wetlands. View south from eastbound I-10 causeway.

Figure 72. Reflooding of Lower 9th Ward by Hurricane Rita surge looking northeast across IHNC toward the reactivated southern I-wall breach. Twin-rotor helicopter in the middle of the picture is dropping bags of rocks into the breach channel that developed.

Figure 73. Reflooding of Southern University of New Orleans campus in Pontchartrain Park by surge through reactivated IHNC breaches during Hurricane Rita.

Timeline for Overtopping and Breaching

The following timetable summarizes the flooding of Greater New Orleans on Monday morning, August 29, with the arrival and passage of Hurricane Katrina:

0400-0500: - Relatively minor breaches at the intersection of the CSX Railroad and the northern arm of the IHNC send water into Orleans East and Orleans Metro to the west. This flooding lasts about 8 hours, and was reactivated in some locations for Hurricane Rita (September 24).

- The MRGO levees in St. Bernard Parish begin to be destroyed by waves, and control structure closures begin to wash out initiating the filling of the St. Bernard wetland buffer.
- Limited flooding into southern St. Bernard over the east bank of the Mississippi River near Poydras.

0500-0600: - Major levee breach into Desire area of Orleans Metro on west bank of the IHNC near France Rd.

- According to IPET there was a major floodwall failure just south of Florida Avenue (north breach) into Lower 9th Ward. ILIT and Team Louisiana believe this breach was at least an hour later.

- New Orleans East Back Levee adjacent to B. Sauvage National Wildlife Area overtops and breaches
- MRGO/GIWW connection from funnel to IHNC overtops north and south levees initiating significant flooding of New Orleans East industrial area, and adding to filling of St. Bernard wetlands buffer, with some pouring in the Lower 9th Ward.

0610: - Hurricane Katrina makes landfall in Louisiana near Buras on west side of Mississippi River mouth

0600-0700: - According to IPET, floodwall failure begins at the Mirabeau site (south) on the London Avenue Canal and at Old Hammond Highway on the 17th Street Canal.
- Overtopping of floodwalls and levees on the east and west bank of the IHNC becomes widespread, affecting all basins.

0700-0800: - South breach into Lower 9th Ward followed by north breach
- According to IPET, floodwall failure begins at the Filmore site (north) on the London Avenue Canal.
- Overtopping of low floodwalls in the vicinity of the Lakefront Airport
- Floodwaters from the MRGO overtop the railroad and Chef Menteur ridge and begin entering the residential areas of New Orleans East from the south.
- Caernarvon to Verret Levee overtopped without damage to the levee.

0800-0930: - Surge on Lake Borgne peaks followed by Lake Pontchartrain south shore 30 minutes later.
- The St. Bernard wetland buffer fills and floodwaters begin cascading over the 40 Arpent Levee into the developed areas of St. Bernard east of Paris Road.
- The 17th Street and London Canal breaches in Orleans Metro reach full development.
- The unfinished wall sections of the Orleans Canal begin overflowing.
- Flooding of New Orleans East residential areas from the east begins as water in the B. Sauvage National Wildlife Refuge overtops the local levee.

0930-1030: - Surge peaks on north shore of Lake Pontchartrain
- Surge peaks on Mississippi coast

PART 2

FORENSICS

The following chapters are devoted to providing answers to the forensics questions below, that guided our investigations. Recommendations for improving future performance are included as the last chapter.

- Was the GNO HPS properly conceived to accomplish the Congressional mandate to protect against the "most severe combination of meteorological conditions reasonably expected?"
- Were the levels of protection, or crown elevations, specified in designs for HPS elements sufficient to resist overtopping by surge and waves?
- Did incorrect design assumptions compromise performance? Should these have been detected and corrected by engineers equipped with the tools available at the time?
- Did the Mississippi River Gulf Outlet (MRGO), a free-flowing, deep-draft navigation canal that pierced the HPS on the eastern side, compromise system performance?
- Was the system maintained and operated to assure the required level of protection through time? Specifically, how did the 40-year construction schedule impact system performance?
- What effect did local cost-sharing sponsors have on HPS design, construction and maintenance, and ultimately on system performance?
- What lessons going forward have been learned from the failures?

CHAPTER FOUR

WAS THE HPS DESIGNED TO FULFILL THE CONGRESSIONAL MANDATE?

Forensics Question 1: Was the GNO HPS properly conceived to accomplish the Congressional mandate to protect against the "most severe combination of meteorological conditions reasonably expected?"

Answer: No, there were a number of fatal design flaws, some crept into the process with time, and some were present at the onset of the GNO HPS

The Design Objective

Although studies had been in progress since 1955, the Lake Pontchartrain & Vicinity Hurricane Protection Project was authorized for construction by the Flood Control Act of 1965 (PL 89-298). This bill was enrolled a month after President Johnson viewed the destruction caused by Hurricane Betsy. Federal appropriations for the initial construction work were made available in May 1967. With this Act, Congress ordered the USACE to protect New Orleans from ***"the most severe meteorological conditions considered reasonably characteristic for that region."*** This language was not coincidental. The USACE had been working with the U.S. Weather Bureau for some time to develop a representation of a hurricane that could set a standard for the design of hurricane-proof structures. Later, the U.S. Nuclear Regulatory Commission and the Federal Emergency Management Agency (FEMA) would join the USACE in funding for this work.

The US Weather Bureau introduced the Standard Project Hurricane (SPH) as "a hypothetical hurricane" that is intended to represent the most severe combination of hurricane parameters that is "reasonably characteristic" of a specific region, excluding extremely rare combinations (Graham and Nunn 1959). Graham and Nunn (1959) further state that:

> "The SPH is intended as a practical expression of the maximum degree of protection that should be sought as a general rule in the planning and design of coastal structures for communities where protection of human life and destruction of property are involved. The height of the storm generated surge is the critical factor. This is mainly dependent on the

> speed and direction of winds as a hurricane nears and enters the coast. Thus, the important product of the study is the hurricane wind field."

The USACE interpreted the Congressional mandate to require a protection system that would prevent flooding by surge and waves associated with an SPH storm. While the SPH was recognized at the time to be less than the **most severe storm or meteorological condition that could strike the city**, it was thought to be all that was justified even for a heavily developed urban area. Perhaps the first mistake?

The Weather Bureau provided storm parameters, including minimum central pressure (CPI), rate of movement (translation) and maximum sustained wind speed at landfall. The USACE had to carry out oceanographic analyses of the combined tide, surge and wave conditions that would be generated by an SPH approaching the city at a range of speeds from all possible directions. This conversion of the wind storm into an oceanographic phenomenon was based on theory, experience with past storms, and the computational technology available at the time.

Finally, engineers had to make allowances for uncertainties and changes that might be expected over the lifetime of the project, particularly subsidence or settling, in order to establish the "design elevation of protective structure," the required crown height for each levee or floodwall reach. Once this elevation was decided, the engineers would choose the structure type, whether a floodwall or levee, and ensure that it could resist all static and dynamic loads with water to the top.

The SPH is often referred to as the "design storm," but this is misleading. First, it is a hypothetical construct rather than any single storm. Second, the storm-generated oceanography sets minimum, rather than maximum, criteria for the crown elevation of levees and floodwalls relative to mean sea level. Over time, however, the SPH increasingly was conflated with the design itself in USACE General Design Memoranda (GDMs) and in the culture of the New Orleans District. Indeed, discussion of the SPH "design storm" continues to permeate the IPET post-Katrina analyses, as we read, for example, that "the surge exceeded design criteria, but the performance was less than design intent" (IPET 2006b, I-3). As we will show, the designers initially understood that the SPH was only one factor to be considered, but this holistic view appears to have been lost later as the SPH became enshrined as the 1965 "authorized level of protection."

What is the Standard Project Hurricane?

Here, we examine the history of the SPH as it evolved after 1959 through a series of reports by the U.S. Weather Bureau and its successor, the National Weather Service. We also look at the way the New Orleans District reported progress on the project through reports of the General Accounting Office -- after 2004 the Government Accountability Office -- abbreviated GAO. The GAO examines the use of federal public funds, evaluates federal programs and activities, and provides analyses, options, recommendations, and other assistance to help Congress make effective oversight, policy, and funding decisions.

By 1959, the U.S. Weather Bureau had developed some options for characterizing severe storm conditions on the northern Gulf coast. These included the "Probable Maximum Hurricane," which theoretically has an almost infinite return frequency, and the SPH, a hypothetical storm believed at the time to have a 1 in 100 year return frequency (1 percent) (Figure 74) in Zone B on the northern Gulf of Mexico. Zone B extends from Pensacola, Florida, to Cameron, Louisiana (Figure 75).

The Weather Bureau was making these determinations about five years before the advent of satellite imaging and only about two years after the first systematic deployment of instrumented hurricane hunter aircraft. Further, the reliable database on landfalling hurricanes at this time extended back less than 60 years to when ships began to radio in offshore reports. This analytical period was only recently extended back another 50 years (Landsea et al. 2002).

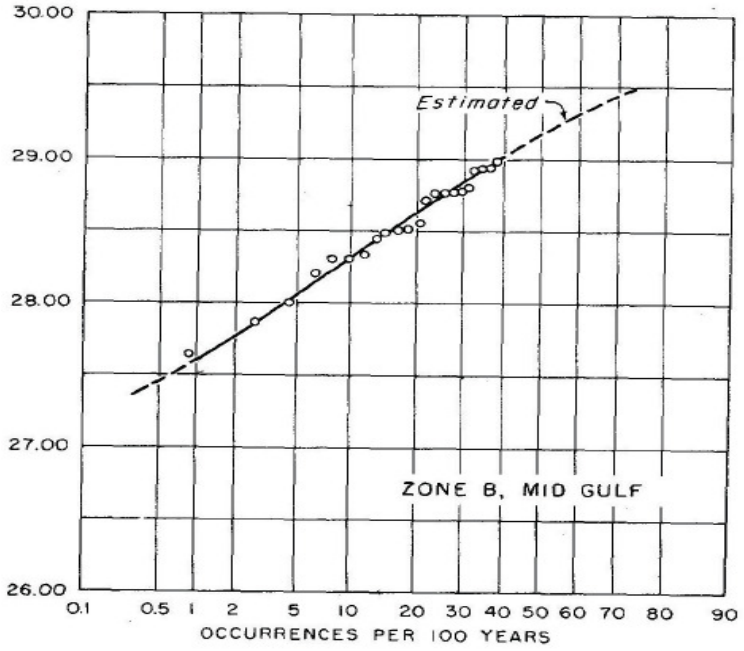


Figure 74. Return frequency for northern Gulf of Mexico storms with a range of minimum central pressures (in inches on vertical axis) from 1959 Weather Bureau publication (Graham and Nunn 1959)

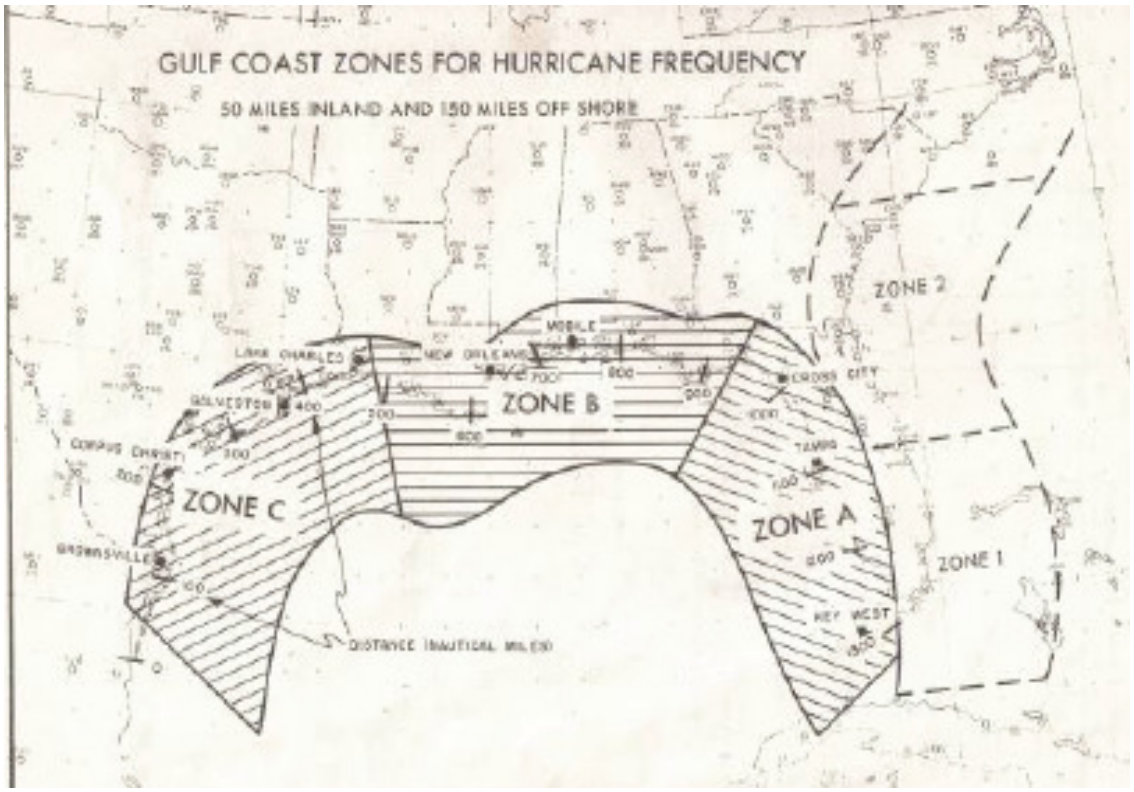


Figure 75. Standard Project Hurricane 1 in 100 year return frequency zones from 1959 Weather Bureau report (Graham and Nunn 1959).

The U.S. Weather Bureau, and its successor the National Weather Service, redefined SPH parameters for the Gulf Coast in 1965, 1968, 1972 and 1979 (Graham and Nunn 1972, Schwerdt et al. 1979) to reflect the increase in experience and understanding regarding extreme hurricanes, particularly Hurricanes Audrey (1957), Carla (1961) and Camille (1969). The categories of the Saffir-Simpson Scale were not introduced until 1971 (Table 5), but can be applied if the 5-minute average wind speeds in the 1959 report and the 10-minute averages in the 1972 and 1979 report are converted to 1-minute averages (Table 5). It is necessary to multiply 10-minute average winds acquired at a 10 meter elevation by 1.14 to obtain the 1 minute average used in Saffir-Simpson. We used a factor of 1.07 to convert the 1959 5-minute average winds to 1-minute averages.

The 1959 SPH was just one stop on a learning curve for the meteorologists. For the USACE, however, the 1959 SPH became the "design storm." Following Hurricane Camille in 1969, the Weather Bureau began a serious re-evaluation that led in 1972 and 1979 to an SPH with maximum sustained winds (1-minute average) nearly 40 percent higher than in 1959. **Clearly, by 1972, at a relatively early stage in the construction of the GNO HPS, the SPH used as a basis for design was obsolete.**

The SPH is a compilation of separately analyzed statistics from many hurricanes. As a result, the CPI is that of a Category 4 hurricane, while in all other particulars it is Category 3 (Table 5) excerpt for the maximum winds in the 1972 and 1979 definitions; that is in 139-140 MPH, a Category 4(Table 6). Hurricane Katrina shared this anomaly on landfall probably because it was weakening from a far stronger storm, and retained a central pressure lower than would be typical for a Category 3 hurricane. Its maximum sustained winds were 10 mph higher than that of the 1959 SPH, but more than 10 mph less than specified in the 1972 and 1979 SPH definitions. The wind speed is critical, because relatively small changes in the maximum winds have big effects on surge, as will be discussed later.

Table 5 Saffir-Simpson Scale Categories of Hurricane Severity

Type	Category	Pressure (mb)	Maximum Sustained Winds 1-min ave. (mph)	Surge (ft)
Tropical Depression	TD	-	<39	-
Tropical Storm	TS	-	39-73	-
Hurricane	1	>980	74-95	4-5
Hurricane	2	965-980	96-110	6-8
Hurricane	3	945-965	111-130	9-12
Hurricane	4	920-945	131-155	13-18
Hurricane	5	<920	>155	>18

SPH Return Frequency Analysis

The NWS uses the arbitrary 100-year return period of the central pressure index (CPI) as a standard for comparison and consistency from zone to zone. However, the meteorologists were careful to point out that the SPH has no frequency assigned to it for assessing whether storm surge will exceed any given elevation at any specified location (NWS 1972). The angle of storm approach to the coastline, the rate of travel, and the radius of maximum wind are important meteorological considerations for determining the hurricane surge. This was certainly true for Katrina that generated a 30 foot surge in Mississippi over a much greater stretch of coast than Hurricane Camille, yet was nominally a storm that fit most SPH criteria (Table 6). Hurricane Camille is generally excluded from SPH analyses as an unusually intense outlier.

Table 6. National Weather Service Standard Project Hurricane in 1959, 1972, and 1979, compared to Hurricane Katrina (Graham and Nunn 1972, Schwerdt et al. 1979, Knabb et al. 2005)

Meteorological Parameter	1959 Weather Bureau Rept 33	1972 NOAA HUR 7-120	1979 NOAA NWS Rept 23	2005 Hurricane Katrina NOAA TCR
Central Pressure Index				
CPI (inches)	27.6	27.4	27.4	27.1
CPI (millibars)	934 (CAT4)	927 (CAT4)	926 (CAT4)	923 (CAT4)
Radius of Maximum Winds (miles)				
Small	8	7	6	
Medium	16	14		
Large	35	25	28	25-30
Forward Speed (mph)				
Slow	5	5	5-12	
Moderate	13	13	12-41	16
Fast	32	32	41	
Maximum 10 m Wind Speed converted to 1-minute average (mph)				
Slow	101 (CAT3)	123 (CAT3)	129 (CAT3)	
Moderate	107 (CAT3)	129 (CAT3)		125 (CAT3)
Fast	111 (CAT3)	139 (CAT4)	140 (CAT4)	

SPH return frequency did not directly influence HPS design. On the other hand, a statistical understanding of the return interval is a basis for assigning risk for insurance purposes, and for communicating this risk to the public. While the SPH has always been a hypothetical 1 in a 100 year storm, the GAO testified before the House Subcommittee on Energy and Water Development, Committee on Appropriations, as recently as September 28, 2005, that the GNO HPS project designs "were developed based on the equivalent of what is now called a fast-moving Category 3 hurricane that might strike the coastal Louisiana region once in 200-300 years" (GAO 2005). This same assertion appears in other GAO reports dating back to 1976 (GAO 1976, 1982).

The basis for such a claim, often repeated by the USACE in New Orleans, is not clear. IPET suggests that a 200 to 300 year return period was derived by subdividing the original 400 mile stretch of coast considered in the SPH to focus on a smaller reach in the immediate vicinity of New Orleans (IPET 2006b, III-34). If such a calculation was made, the likelihood of an SPH striking any 50 nautical mile reach of coast within Zone B would be more like 1 in 500 years. This illustrates one of the problems with return frequency statistics that plagues hurricane science to the present. Because hurricanes rarely strike exactly the same place repeatedly, if a small enough target coast is defined, it is possible to arrive at very low return frequencies, and be lulled into complacency. The claim of 1 in 200 or 1 in 300 year protection for the GNO HPS created a false sense of security at a time when alarms should have been sounded.

In 1987, NOAA conducted a detailed analysis of hurricane climatology that included dividing the Atlantic and Gulf coasts into 10 nautical mile segments and assessing the return frequency for tropical storms and hurricanes within each segment (Ho et al. 1987). Although New Orleans fell within a relatively high zone of return frequency, less than two tropical storms of any intensity made landfall over the past century in any 10 nautical mile segment that included the GNO (Figure 76). Different statistical methods have been developed over time but they all involve extrapolation from a limited storm database. NOAA has recently extended this database back to the 1850s based on careful research of contemporary news accounts (Landsea et al. 2002), but all return frequency statistics assume that the last century is a good predictor of the future. In an age of global warming and storm intensity cycles lasting for decades, it almost certainly is not.

The actual record of hurricane landfalls in Louisiana provides a reality check on both the viability of the SPH approach and the various statistical return frequency methodologies (Table 7). With a slight modification in track, any of these storms could have hit New Orleans far worse than it did. Since 1899, 30 hurricanes have made landfall in Louisiana and 14 of these were major storms - Category 3, 4 and 5. This equates to a major storm striking the Louisiana coast once every 7.6 years. Nine of these storms caused damage in the GNO area, for a return frequency of once every 12 years.

If we look more closely at the nine major storms (>Category 2) that have affected the GNO since 1899, the list includes one Category 5, two Category 4 and six Category 3 hurricanes (Table 8). One Category 5 hurricane is expected to hit New Orleans every 180 years. Similarly, Category 4 and 3 storms will strike the city once in 70 and 32 years, respectively. Lesser Category 2 and 1 hurricanes can be expected every 14 and 8 years, respectively.

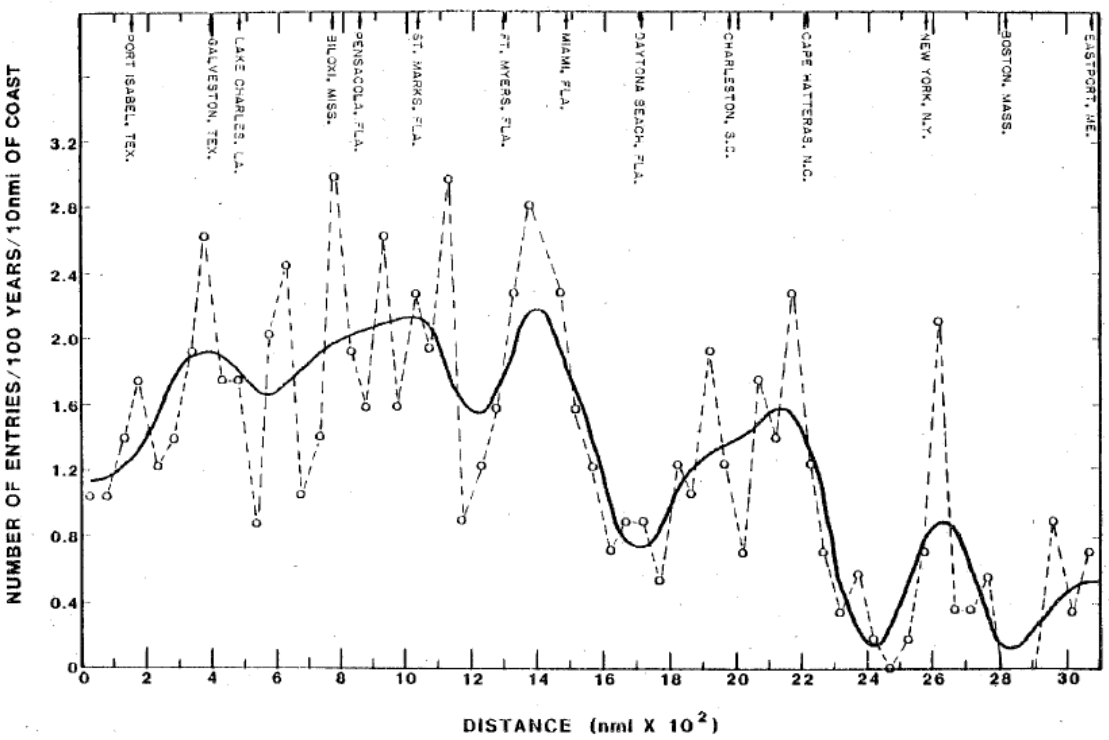


Figure 76. Frequency of landfalling tropical cyclones for the 1871 to 1984 period for any 10 nautical miles of the smoothed Gulf and Atlantic coasts. New Orleans occurs at mile 700 on this figure in a zone of relatively high return frequency, but still has experienced less than 2 tropical storms of any intensity per century within any 10 nautical mile segment (From Ho et al. 1987).

Table 7 Hurricanes (direct hits) affecting Louisiana 1899-2005
(modified from http://www.srh.noaa.gov/lch/research/tropical5.php)

	Number of Hurricanes: Saffir/Simpson Category Number						Landfall frequency (storms per year)	Return Period (years)	Coastline Length (in st. mi.)	Distance Between Landfalls (in st. mi.)
	1	2	3	4	5	Total				
Louisiana (LA)	11	5	10	3	1	30	0.27	3.7	397	13.2

Table 8. Major Hurricanes Affecting the GNO 1893-2005

Hurricane	Year	Category	CPI (mb)	Max Sustained Winds (mph)
Camille	1969	5	909	200
1979 SPH	1979	¾	927	140
1972 SPH	1915	4	931	140
1909 Storm		*3/4*	*927*	*129*
Katrina	2005	3	952	120
Rita	2005	3	923	125
Hilda	2005	3	935	115
1959 SPH	1964	3	962	115
Betsy		*3/4*	*935*	*107*
1947 Storm	1965	3	948	105
1893 Storm	1947	3	955	100
Andrew	1893	3	955	115
	1992	3	956	96

Four of the severe hurricanes that have affected the GNO area over the past 110 years had central pressures lower than the 1959 SPH, while six made landfall with a higher maximum sustained wind speed. So the 1959 SPH would have been a severe storm, but should not have been considered ***"the most severe meteorological conditions considered reasonably characteristic for that region."*** More than half of these 10 storms made landfall before the 1965 authorization for the HPS, and provided a clear indication that the 1959 SPH was deficient. The 1972 and 1979 SPHs are placed on this list for comparison and clearly would have been a better choice. Both the 1972 and 1979 SPHs were available – and ignored -- before most construction on the HPS had proceeded to a point of no return (Table 8).

Does the once in a century SPH define a 100 year project life?

Most engineering works have a defined project life. Like a structure that is designed to protect against floods up to the crown elevation, a period is defined within which it is assured that the system will operate as designed. Design assumptions may become invalid with time, and materials performance cannot be assured beyond the design life. Nowhere in any of the reports issued to date is there any discussion of the design life of the GNO HPS. Clearly, a project which has no specified path for upgrade must have a finite design life, and in lieu of any other evidence, it appears that the 100-year span of

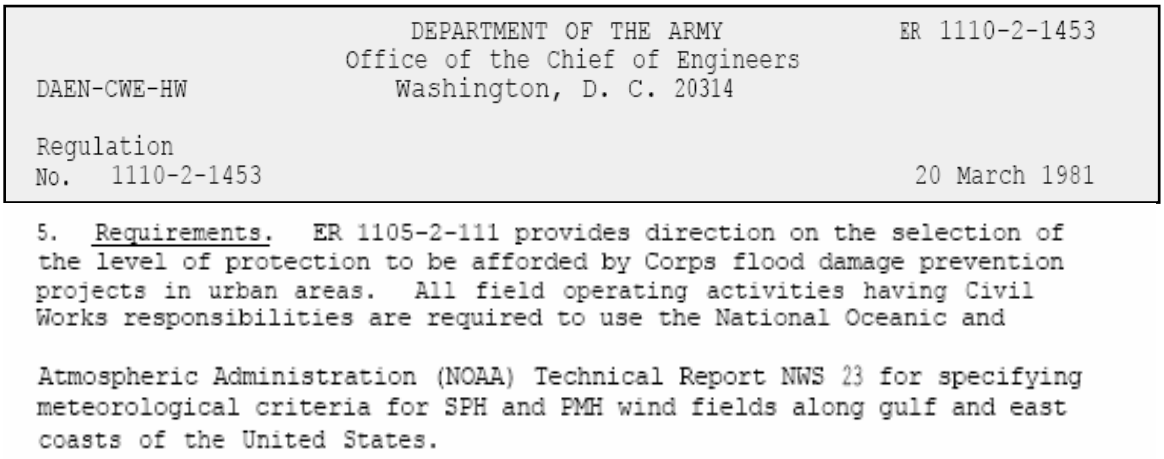
the SPH statistic provides the only indication. If this is the case, then the GNO HPS was designed to provide protection for 100 years.

The question then becomes, when does the 100 year performance period begin? Does it start with authorization, or upon completion of all construction? If the later is true, then some elements of the system must be designed for a longer period of performance, perhaps up to 150 years for some of the earliest elements of the GNO HPS.

An Opportunity Missed: USACE Policy on Use of the 1979 SPH

The Chief of Engineers distributed an order in March, 1981, to all districts with urban hurricane flood damage prevention projects under construction or in planning (Figure 77). The order directed that all such projects be designed using the SPH as it had been redefined in the 1979 NWS 23 report (Schwerdt et al. 1979). The 1979 SPH was a slight refinement of the 1972 version (Table 7). Many GDMs were finalized after this date, but there is no evidence that the New Orleans District ever complied with the 1981 order on any portion of the East Bank GNO HPS.

The only "design hurricane" calculations of surge and waves applied to the East Bank GNO HPS appear to have been based on the 1959 SPH, and the design calculations all appear to have been made prior to authorization. **The 1959 SPH was used to develop the preliminary information upon which original cost estimates in the authorization were based (Table 9). The observed surge from Hurricane Betsy was found to be lower than was estimated using the earlier analysis, and this was taken as a validation of the original approach, and a reason to proceed without modification (IPET 2006b, III-40).** This unrealistic and simplistic approach then dominated all further levee designs, without question.



DEPARTMENT OF THE ARMY ER 1110-2-1453
Office of the Chief of Engineers
DAEN-CWE-HW Washington, D. C. 20314

Regulation
No. 1110-2-1453 20 March 1981

5. Requirements. ER 1105-2-111 provides direction on the selection of the level of protection to be afforded by Corps flood damage prevention projects in urban areas. All field operating activities having Civil Works responsibilities are required to use the National Oceanic and

Atmospheric Administration (NOAA) Technical Report NWS 23 for specifying meteorological criteria for SPH and PMH wind fields along gulf and east coasts of the United States.

Figure 77. Excerpt from USACE Headquarters regulation requiring updated SPH for all existing or planned coastal projects in urban areas.

One GDM being developed at the time the 1981 order was received by the New Orleans District from USACE headquarters was the "Jefferson Parish St. Charles Parish Return Levee" Design Memorandum (No.17A General Design). It was first issued in July, 1987, and provides designs and specifications for construction of the Duncan Canal floodwall along the west side of the Jefferson Parish basin, from Lake Pontchartrain south to Airline Highway near the Mississippi River (See Figure 7a). Six years after it was mandated by USACE headquarters, the original 1959 SPH parameters are invoked and there is no indication of a migration toward the higher maximum wind speed of the 1979 SPH.

Two years before the GNO HPS was originally scheduled for completion, in 1976, the Comptroller General of the United States reported on the progress of the New Orleans levee protection plan (http://archive.gao.gov/f0402/098185.pdf). This GAO report points out that only 27 of 73 plans and specifications had been issued, and, further, that the New Orleans District was not spending its full annual allotment for the project despite the assertion that it was the top priority. The 1976 report also noted that USACE was requesting an additional "$22 million for increasing the size of the levees based on better definition of the standard project hurricane as furnished by the United States Weather Bureau."

It is not clear what happened to this request or why the analysis backing this request was never incorporated in the many design memoranda then under development. **What is important here is that as early as 1976, the USACE knew that the original SPH-based design criteria were deficient.**

Sensitivity of Surge to Maximum Sustained Wind Speed

Three hypothetical critical hurricane tracks were developed, called synthetic tracks, as a step in the original oceanographic analysis. SPH storms following these tracks were used to generate the theoretical surge and wave conditions for the various HPS floodwall and levee reaches (Figures 78 and 79). Synthetic track 'A' can be seen to trace a path similar to that actually followed by Hurricane Katrina (see Figure 1), while the speed of translation is also similar (Table 9).

All of the GNO HPS GDMs include the following statement.

> "Three observed storms, with known parameters and effects, were used to establish and verify procedures and relationships for determining surge heights, wind tide levels (WTL's), inflow into Lake Pontchartrain,

overtopping flows, and ultimately flood elevations that would result from synthetic hurricanes."

Table 9. Lake Pontchartrain and Vicinity HPS Design Hurricane Characteristics (IPET 2006b, III-38, Table 5; III-160, Table 14)

Project Location	Track	CPI (inches)	Radius Maximum Winds (miles)	Speed of Translation (mph)	Maximum 1-min ave. Wind Speed (mph)	Direction Of Approach
Lake Pontchartrain Southshore	A Fig.5.5	27.6	35	7	107	S
Lake Borgne, Rigolets & Chef Menteur Passes	F Fig.5.5	27.6	35	13	107	E
Chalmette Area & Ext. Along MRGO	F Fig.5.6	27.6	35	13	107	E
IHNC East	F Fig.5.6	27.6	35	13	107	E
Chalmette Ext.	C Fig.5.6	27.6	35	6	107	SSE

The storms chosen for the Lake Pontchartrain HPS elements made landfall in 1915, in 1947 and in 1957. The 1915 and 1947 hurricanes appear in the list of major storms that have affected the GNO area, and the 1915 hurricane appears to have been a Category 4 storm of comparable intensity to the 1972 SPH (Table 8). The 1947 hurricane was a Cat 1 when it reached the GNO (Yamazaki and Penland 2002).

But the 1957 storm, Esther, that was used for validation, was not a hurricane, but a tropical storm with maximum sustained winds of about 50 mph. Hurricane Flossy, a Category 1 storm that made landfall in 1956 and passed well to the east of the city, was substituted for TS Esther to validate surge estimates for the GNO area east of the IHNC (Figure 78). Hurricane Betsy, the 1965 storm that caused more damage to the GNO than

any storm prior to Katrina, does not appear in the design discussion because the oceanographic analysis was completed long before Betsy arrived. No recognition is present that if Category 5 Camille had moved 40 miles to the east that catastrophic surges would have impacted the GNO. What missed opportunities to re-evaluate the original designs! What is also glaringly missing from the NOD analysis is a recognition that rapid wetland loss was one of the main features of the Louisiana coastal landscape.

In its pre-1965 oceanographic analysis, the USACE used a relatively simple general wind-tide equation to develop surge estimates (IPET 2006b, III-39):

$$S = 1.165x10^{-3} Z(V^2F/D) \cos \theta$$

where,

S = wind setup (surge) in feet
V = wind speed (mph)
F = fetch length (miles)
D = average depth of fetch (feet)
Θ = angle between direction of wind and the fetch
Z = surge adjustment factor

The surge adjustment factor, 'Z', was empirically determined based on high water mark data from the 1915 and 1947 hurricanes, and was validated against similar data collected after passage of the two lesser storms in 1956 and 1957. It was determined to range from 0.2 to 0.5 for different reaches of the HPS. Adjustments were also made east and west of the city for a frictional reduction of surge due to flow over the marsh (Figure 80). A reduction of 1 ft for each 2.7 miles of marsh fronting a levee or floodwall had been derived from analysis of high water marks in southwest Louisiana for a number of storms prior to 1963. Mashriqui et al. (2007 – in preparation) recently found, based on data also collected in southwest Louisiana after Hurricane Rita, that this value is acceptable for marshes distant from large dredged channels. So, the USACE was aware of the value of the wetlands, yet made no adjustments for the ravages of wetland loss.

The only term raised to a power in the general wind tide equation is 'V', the wind speed. As a result, it has a disproportionately large effect on surge. If all other factors are kept constant, for example, an increase in wind speed from 105 to 115 mph (9.5 percent) could add an additional 3 feet to a 15 foot surge on the MRGO levee, for a 20 percent increase.

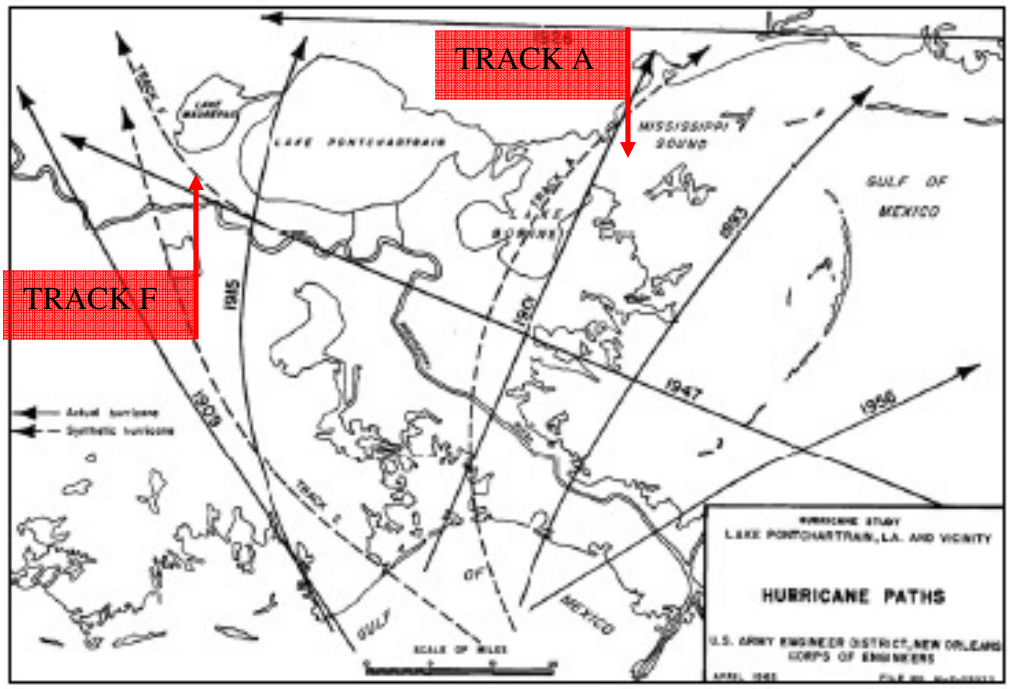


Figure 78. Paths of actual hurricanes affecting New Orleans contemplated for USACE design, as well as critical synthetic paths used in oceanographic analysis for the Lake Pontchartrain part of the design (IPET 2006b, III-38). Synthetic track 'A' is similar to that followed by Hurricane Katrina.

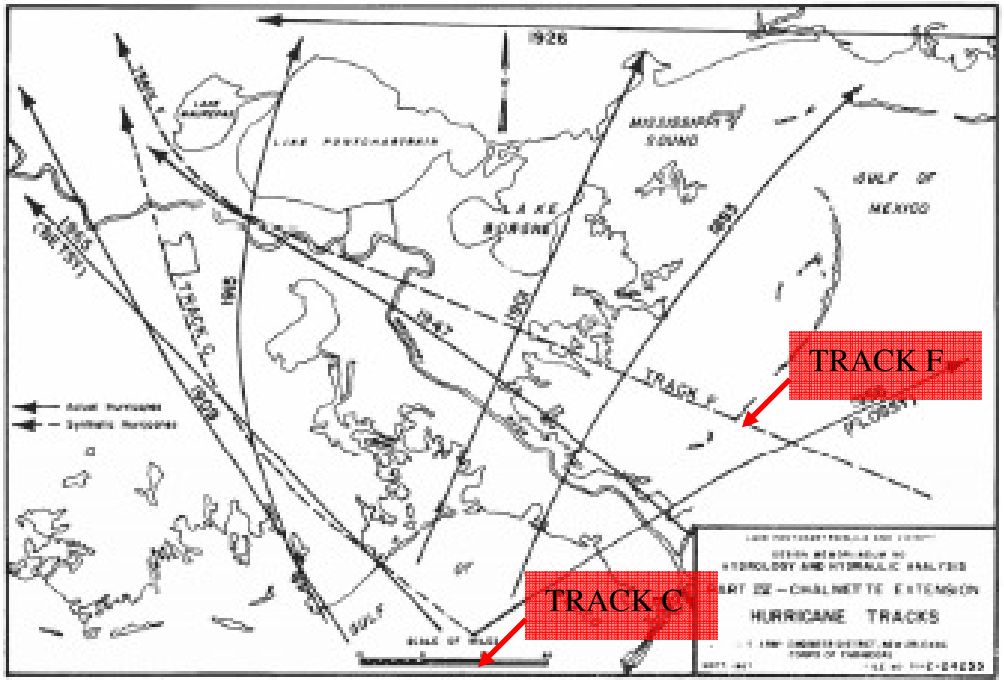


Figure 79. Paths of actual hurricanes affecting New Orleans contemplated for USACE design, as well as critical synthetic paths used in oceanographic analysis for area east of IHNC (IPET 2006b, III-161).

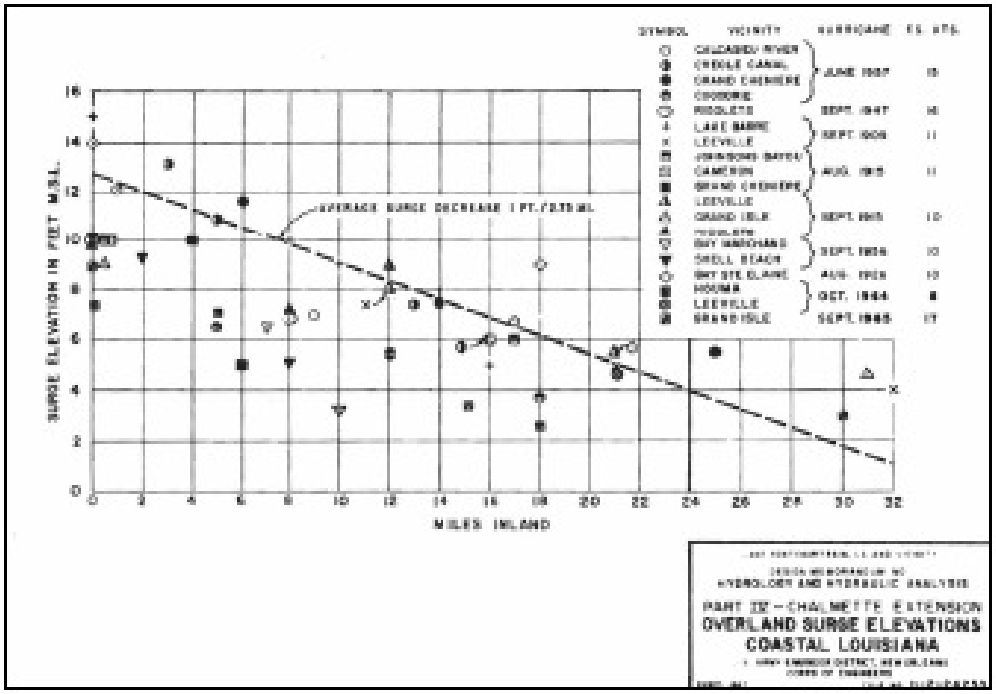


Figure 80. Empirical relationship of surge reduction associated with friction during flow over wetlands that were used to reduce levee and floodwall crown height requirements in elements of the GNO HPS that faced extensive wetland areas. The line depicts an "average surge decrease" of 1 ft for every 2.7 miles of overland flow.

IPET obtained a similar result when testing the sensitivity of the ADCIRC surge output to 5 percent increases or decreases in wind speed. A 5 percent reduction or increase in wind speed in the vicinity of the MRGO levee resulted in a 1.5 foot change in the surge elevation (IPET 200b, IV-140,141). The difference between the maximum wind velocities associated with the 1959 and 1972 SPHs, 107 and 129 mph, respectively, is 21 percent. **If all other variables are again held constant, the general wind tide equation predicts that a 21 percent increase in wind velocity would cause surge to increase on the MRGO levee by 6.5 ft, or 41 percent (Figure 81).** The differences between the 1959 and 1979 SPHs would have resulted in a significant increase in expected SPH surge heights. Most importantly, the NOD admitted in 1976 to the GAO that they needed to raise levee heights because of the 1972 SPH change, but then appear not to have taken action.

Another Opportunity Missed: the Advent of Numerical Surge Modeling

Scientists had been using computers to run increasingly sophisticated numerical models to predict hurricane surge for many years when Chester Jelesnianski of the U. S. Weather Bureau developed the "Special Program to List Amplitudes of Surge from Hurricanes," or SPLASH model (Jelesnianski 1972). This model scored an immediate

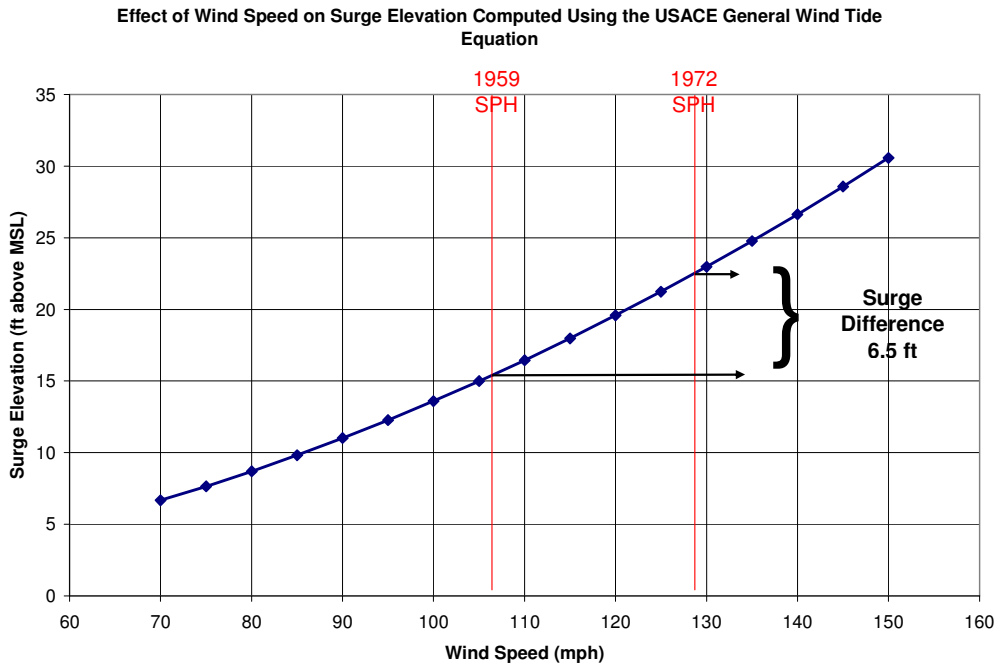
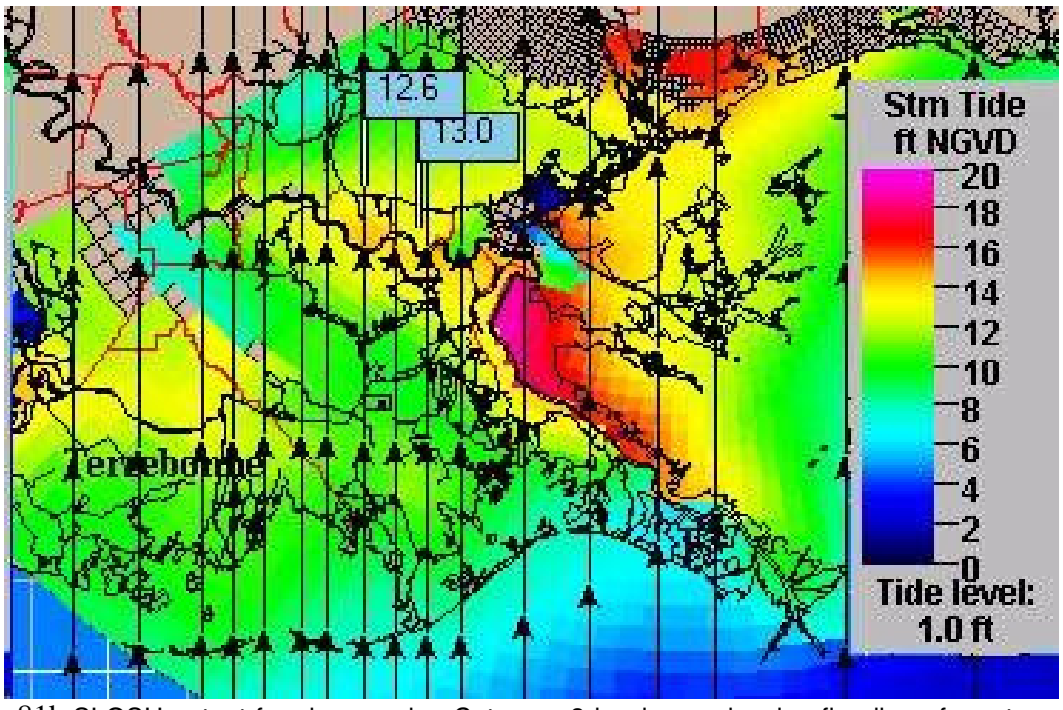


Figure 81a. Effect of wind speed on surge elevation on the MRGO levee if all other factors in the general wind tide equation are kept constant. The red lines indicate the maximum sustained winds associated with the 1959 and 1972 Standard Project Hurricanes.



81b.SLOSH output for slow moving Category 3 hurricane showing flooding of most of greater New Orleans (source NOAA).

triumph by accurately hindcasting the devastating surge that accompanied Hurricane Camille in 1969 (van Heerden and Bryan 2006). Jelesnianski then developed SLOSH (Sea, Lake, and Overland Surges from Hurricanes), which was first released in 1979 and is still in use by the National Oceanographic and Atmospheric Administration and other agencies (Jelesnianski et al. 1992). So SLOSH became available at about the same time as the NWS 23 revision of the SPH (Schwerdt et al. 1979).

The SLOSH model computes water height over a geographical area or basin (Figure 81b). Computations have been run for a number of basins covering all of the Atlantic and Gulf Coasts of the U.S., as well as the offshore islands. The typical SLOSH grid contains over 500 points located on lines radiating from a common basin center. Bathymetric and topographic map data are used to determine a water depth or terrain height for each grid point. The coastline is represented as a physical boundary within the model domain. Sub-grid-scale water features (passes and channels), and obstructions (levees, roads, spoil banks) have been parameterized within the model. Astronomical tides, rainfall, river flow, and wind-driven waves have not been incorporated, however, so SLOSH predicts only surge.

The primary use of SLOSH model is to define flood-prone areas for evacuation planning. The flood areas are determined by compositing model surge values from 200-300 hypothetical hurricanes to form a Maximum Envelope of Water, or MEOW chart. Separate composite flood maps are produced for each of the five Saffir-Simpson hurricane categories, and for slow, medium and fast-moving storms. The SLOSH model can also be run using forecast track and intensity data for an actual storm as it makes landfall. The USACE and FEMA note on their joint "Comprehensive Hurricane Data Preparedness Study Website:"
(http://chps.sam.usace.army.mil/USHESDATA/SLOSH/sloshgeneral.htm)

> "In addition to computing surge heights for the open coast, the SLOSH model has the added capability to simulate the routing of storm surge into sounds, bays, estuaries, and coastal river basins, as well as calculating surge heights for overland locations. Significant natural and manmade barriers are represented in the model and their effects simulated in the calculations of surge heights within a basin."

The Corps helped fund the development of SLOSH, and later ADCIRC, and conducted surveys around New Orleans so that the topography and bathymetry could be presented with an appropriate level of detail. The 1979 version of SLOSH predicted that a Category 5 storm could generate a surge of more than 20 feet along the south shore of Lake Pontchartrain. Other runs in the 1980s indicated that the GNO HPS was

insufficient to prevent overtopping by a wide range of Category 3 hurricanes. While there was nearly universal agreement in the scientific community that SLOSH was a superior tool in many ways for predicting the oceanographic response to hurricane winds, there is no indication that the Corps ever used the SLOSH model in developing GDMs for the HPS.

The GAO was more critical of the USACE in a 1982 report than it had been in 1976, noting that estimated costs to complete the GNO HPS had escalated by a factor of ten and were now close to $1 billion, while the originally authorized work was only 49 percent complete after 15 years (http://archive.gao.gov/d42t14/119206.pdf). The increases in costs were largely attributed to foundation problems discovered after the work began, but were also attributed to the need to raise levees, particularly along the drainage canals to meet the "Weather Bureau's new data pertaining to hurricane severity." It is not clear what steps, if any, the USACE took to address the deficiency in maximum sustained wind speed associated with the pre-1965 SPH-based analysis. While the USACE offered many excuses about the slow progress and cost overruns, it does not appear that they ever raised alarms about the inadequacy of the level of protection that they were, in fact, expecting to provide. The report quotes statements by local sponsors that they believed that the work would not be expedited until another tragedy struck the city. These statements, sadly, have been proved correct.

Coastal wetland loss in Louisiana and its significance

The scientific community had been raising the alarm about Louisiana's disappearing wetlands since the late 1970's. The Corps was obviously aware of this problem because in September 1986 the chief of the Engineering Division of the New Orleans district wrote the commander and director of the Waterways Department Station requesting funding to undertake an assessment of the wetlands loss. Specifically they investigated land loss between 1932 and 1983. July 1987 the Corps released its report "Geological Investigation of the Mississippi River Deltaic Plan- Land Loss and Land Accretion", authored by J. May and L. Britsch. This excellent reports shows in detail, just how devastating the land loss had been since 1932.

This action proves that the NOD was fully aware of the land loss problem and surely the consequences for reduction in surge protection. But nowhere in the GDM's does the NOD discuss the need to give extra free board to levees and floodwalls because of the worsening land loss. The land loss research ordered and paid for by the USCAE was ignored by those responsible for the GNO HPS.

Summary Findings Regarding the Adequacy of the Original SPH Design Concept

Forensics Question 1
Was the GNO HPS properly conceived to accomplish the Congressional mandate to protect against the "most severe combination of meteorological conditions reasonably expected?"

Answer 1
No. The initial meteorological and oceanographic analysis based on the 1959 U.S. Weather Bureau 1 in 100 year Standard Project Hurricane (SPH) was known to be obsolete by 1972, just as construction of the GNO HPS was getting underway.

The primary deficiency of the 1959 SPH was in the specification of maximum sustained wind speed, which the NWS had increased by 20 percent, from 107 to 129 mph in 1972, and to 140 mph in 1979. The steady-state analytical approach used by the USACE to develop surge estimates was as sensitive to the effect of wind velocity as later numerical modeling approaches (i.e. SLOSH or ADCIRC), and should have alerted the USACE to the danger of underestimating wind speed. This analysis provided a design basis for setting the minimum heights above mean sea level for levee and floodwall crowns to resist overtopping by combined SPH waves and surge.

A 20 percent underestimate of maximum winds can lead to a 40 percent reduction in the predicted surge elevation. The New Orleans District USACE was aware of this deficiency in the original analysis, as is indicted by testimony in the 1976 and 1982 GAO reports, but never revised the original SPH-based analysis to reflect the new understanding of threat even after being ordered to do so by the Chief of Engineers in 1981 (ER 1110-2-1453). New Orleans residents were not advised that the GNO HPS required significant improvements to meet 1 in 100 year SPH requirements, but, instead, the New Orleans District claimed at times that the GNO HPS would protect against a 1 - in -200 to 1- in -300 year hurricane. No basis for this claim has been established, while numerous storms that have affected the GNO area – before and after the 1965 HPS -- were more severe than the 1959 SPH.

The New Orleans District USACE missed opportunities to revise the original SPH-based analysis after the National Weather Service revised the SPH in 1972 and 1979, and when the SLOSH storm surge model came into use in 1979. SLOSH showed clearly that the GNO HPS, as it was constructed at the time, was vulnerable to overtopping by many possible Category 3 storms. This result was confirmed later by the ADCIRC model. The USACE supported development of both surge models and was aware of GNO HPS vulnerabilities, but appeared to accept the inadequacy of the system with a

complacency that undercut efforts to sound alarms and begin pressing for improvement. The surge increasing consequences of coastal Louisiana's dramatic land loss did not figure into any surge calculations or levee designs- a case of 'ignore the science at your own peril'.

CHAPTER FIVE

WAS THE LEVEL OF PROTECTION INCORPORATED IN DESIGNS ADEQUATE?

Forensics Question 2: Were the levels of protection, or crown elevations, specified in designs for HPS elements sufficient to resist overtopping by surge and waves associated with the 100 year Standard Project Hurricane?

Answer: No. Floodwall and levee crown elevations were built 1 to 2 ft low because of an erroneous assumption at USACE New Orleans District (NOD) that an elevation of zero referenced to the National Vertical Geodetic Datum of 1929 (NGVD29) was equal to -- and interchangeable with -- local mean sea level (LMSL). LMSL was the relevant datum for superimposition of hurricane surge and wave height. In 1965, zero NGVD29 was between 1.3 and 1.6 feet below LMSL, and floodwalls and levee crowns were constructed lower by this margin. This mistake was locked in for continuing HPS construction when the NOD adopted a policy in 1985, with the approval of the USACE Lower Mississippi Valley Division (LMVD), to explicitly use the outdated 1965 NGVD29 adjustment for elevation control, and to ignore subsidence and sea level rise. As a result, no provision was made to account for the 3 to 4 ft/century subsidence rates characteristic of the GNO area even though this rate was known at the time of authorization.

Crown elevation deficiencies ranging up to 5 feet at the time Katrina struck resulted in prolonged overtopping of floodwalls and levees along the IHNC and to the east in the Lake Borgne funnel that otherwise would have been overtopped only briefly. Prolonged overtopping led to catastrophic breaches into the Lower 9th Ward on the east and into Orleans Metro on the west, and contributed to the early failures of levees along the GIWW and MRGO. Early failure of the MRGO levee allowed the 32,000 acre wetland buffer to fill and overtop the 40 Arpent back levee while the surge was still rising, and resulted in catastrophic flooding in St. Bernard to an elevation of 11 ft (NAVD88).

Design Elevation of Protective Structure

Team Louisiana has paid more attention to the "design elevation of protective structure" than to the Standard Project Hurricane (SPH) criteria discussed in the last chapter. From a forensics standpoint, what really matters is how the combined elevation of surge and waves compares, first, with the crown height of a particular levee or floodwall specified in the design, and, second, to the actual crown elevation when that structure received the storm on August 29, 2005. The design crown elevations were higher for some structures than the minimum necessary to prevent overtopping by the SPH. The levee and floodwall crown elevations measured by surveyors post-Katrina were often significantly lower than the design elevations. **If the design elevation is a statement of the level of protection authorized by Congress, then it is important to understand whether the floodwalls and levees were built to a lower elevation than specified, or just ended up low after many years of settlement.** Both of these are issues that must be addressed in the original design. Settlement and sea level rise are predictable.

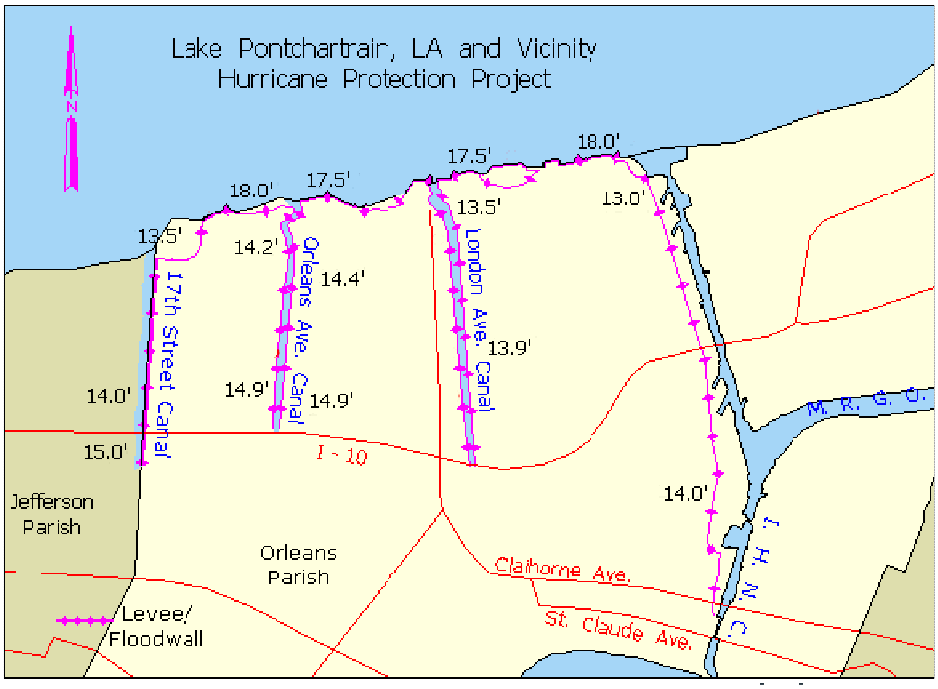


Figure 82. Pre-Katrina USACE representation of HPS levee and floodwall crown elevations for the Orleans Metro basin. Elevations relative to NAVD88 (IPET 2006b, III-36).

All of the post-Katrina investigating teams were initially baffled by the almost universal, and, in some cases, quite substantial deficiencies between the elevation of levee protection specified in the designs and what was actually discovered when earthen levee crest and floodwall heights were surveyed after the storm. Pre-Katrina

USACE charts of the Orleans Metro HPS, for example, give the elevation of drainage canal floodwall crowns as ranging from 14 feet near Lake Pontchartrain to 15 feet near the interior pump stations (Figure 82).

Louisiana Department of Transportation and Development (LDOTD) surveyors found top of wall elevations on both sides of the 17th Street Canal near the breach to be between 12.6 and 12.8 feet (Figure 83) relative to NAVD88. Although these were some of the most recently constructed elements of the HPS, having been built in the mid-1990s, the tops of these walls were 1 to 2 feet lower than they should have been (14.4 ft).

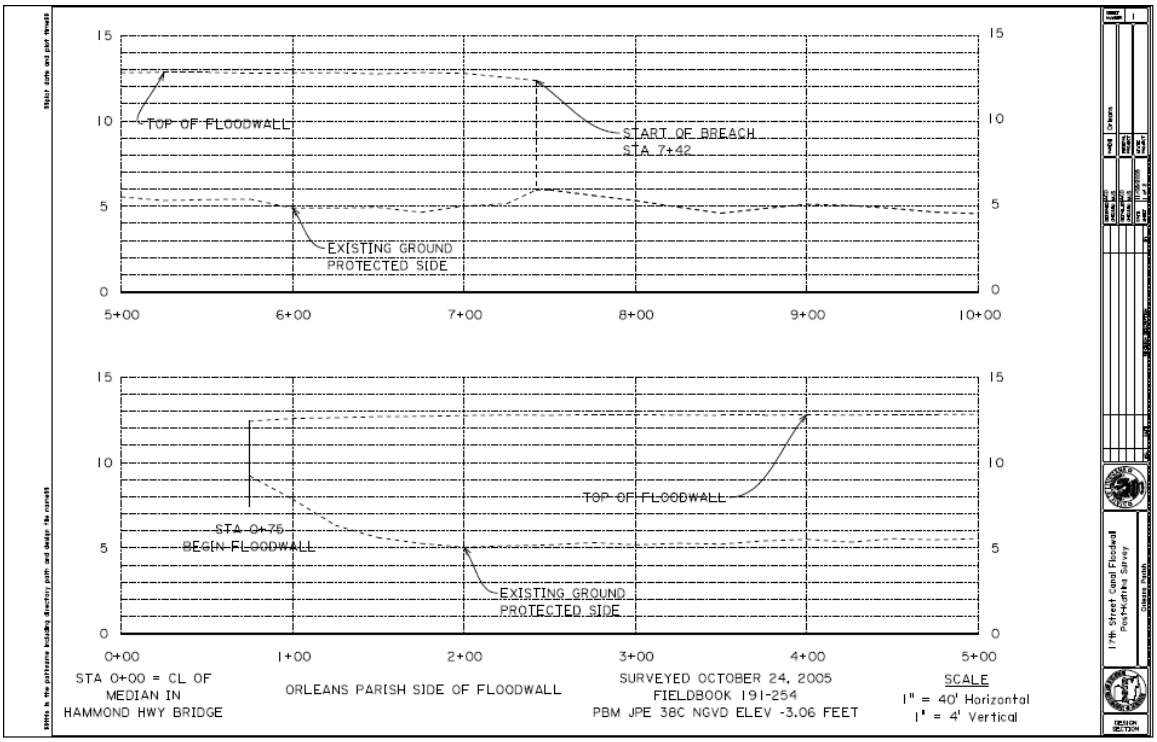


Figure 83. Post-storm LDOTD survey of Orleans Parish floodwall crown elevation wall segments on both sides of the 17th Street Canal breach, showing undisturbed top of wall at 12.8 ft (NAVD88), 12.3 ft relative to current MSL, or 12.5 relative to MSL in the 1990s when it was built. The design level of protection was 14.0 ft MSL. The wall was built 1.5 ft low because an outdated 1965 benchmark adjustment in NGVD29 was used to control construction, rather than local mean sea level (LMSL).

GNO HPS Crowns Built too Low

There is a very simple reason for near universal deficiencies observed on floodwall crest levels. Most levee crown levels are also low, but these structures are more likely to be affected by local settling. In examining design and construction contract documents and drawings, IPET noted that the New Orleans District (NOD) constructed virtually all structures to the elevation specified in the SPH-based design, but relative to the National Geodetic Vertical Datum of 1929 (NGVD29), rather than to the local mean sea

level (LMSL) elevation (IPET 2006b, II). A geodetic datum is a land-based reference system. It is not explicitly connected to the oceanographically relevant mean sea level surface upon which must be superimposed the SPH-based levels of tide, surge and waves discussed in the last chapter. In 1965, when the HPS was authorized, the mean level of Lake Pontchartrain was more than 1.3 ft above 0.0 ft NGVD29 in the GNO area.

Geodetic Datums, Tidal Datums and Adjustments

The National Geodetic Survey (NGS) branch of the National Oceanic and Atmospheric Administration (NOAA) develops and maintains the North American Vertical Datum of 1988 (NAVD88), the land surveyors' standard reference datum since the 1990s. The NGS changed the geodetic reference system from NGVD29 to NAVD88 in 1991 because thousands of bench marks had been destroyed, and many others had been affected by crustal motion, going up in the north due to postglacial rebound, and down in most other places due to loading or subsidence. Substantial distortions had been observed as a consequence of forcing the more than 600,000 kilometers of leveling carried out since 1929 to fit the NGVD29 height values.

The Sea Level Datum of 1929 was originally related to mean sea level observations at 21 tide gages in the United States and 5 in Canada, but none in Louisiana. Satellite altimetry had shown that mean sea level varies in a complex manner from one coastal location to another, and does not form a uniformly level surface as was once thought. For this reason, the NGS changed the name in 1973 to NGVD29 to reduce the chance that anyone would confuse it with mean sea level.

NAVD88 was established by the minimum-constraint adjustment of Canadian, U.S. and Mexican leveling observations. It held fixed the height of a single primary tidal benchmark at Father Point/Rimouski in Quebec, Canada. Additional tidal benchmark elevations were not used and no connection with sea level is implied. NGVD29 and NAVD88 are, like all geodetic datums, somewhat arbitrary reference systems set up to allow internally consistent comparisons of land elevation anywhere on the North American continent. The NGS provides the relationships between past and present geodetic vertical datums.

Mean sea level is unique to a specific location and epoch, or averaging period. It moves over time independently of any geodetic system tied to benchmarks on land, which may also be in motion. The dynamics of sea level change over time create a unique problem for a hurricane flood protection system, because these changes, while relatively slow, must be factored into the design in order to provide a specified level of protection over the lifetime of the project, particularly a project with a 100 year design life (See

Chapter Four discussion, "Does the once in a century SPH define a 100 year project life?")

The National Ocean Service (NOS), another NOAA branch, is responsible through the Center for Operational Oceanographic Products and Services (CO-OPS) for publishing tidal benchmark information and the relationship between NAVD88 and various water level/tidal datums (e.g., Mean Lower Low Water, Mean High Water, Mean Tide Level, etc.). The relationships to the older datum, NGVD29, are not published but may be calculated independently from specified tidal benchmark sheet links to the NGS data base. Tidal bench mark information, water level/tidal datums, and their relationship to geodetic vertical datums are available at the CO-OPS web site (www.co-ops.nos.noaa.gov). All mean sea levels are local to a particular gage, and are therefore designated local mean sea level or LMSL, typically tied to a particular measurement epoch (i.e. 1960-78).

Because individual benchmarks move over time relative to a fixed datum, the NGS conducts leveling studies and periodically publishes adjustments for benchmark elevations. Surveyors typically provide elevation control for construction projects from the most recently published adjustments, unless ordered to do otherwise. This poses another unique problem for a flood protection project like the GNO HPS that was constructed over a nearly forty year interval, and might be expected to have a 100 year project life. If benchmarks are subsiding, as they generally do in the GNO area, elements of the system that are designed to have the same crown elevation, but are constructed later, may end up lower than adjacent segments constructed earlier, even if the same benchmarks are used for control. In 1985, the New Orleans District developed a creative solution to this dilemma to which we will return later.

Top Elevation Deficiencies due to Datums on Orleans Metro Drainage Canals

IPET collected benchmark adjustments along the south shore of Lake Pontchartrain in the vicinity of the mouths of the 17th Street, London Avenue and Orleans Canals (Tables 10, 11, 12). Three epochs of local mean sea level are shown referenced to various geodetic adjustments in NGVD29, and later in NAVD88. The NGS developed a time-stamped NAVD88 adjustment, designated NAVD88(2004.65), specifically to support the IPET investigation of the GNO HPS performance. Benchmark ALCO near the 17th St. Canal was apparently not used as control for the construction of the I-wall structures, but is similar to nearby benchmarks that were (Table 10). It has the advantage of also being a NOAA tide gage reference. Benchmark P-153 was used to control construction of the London Avenue floodwall (Table 11), while benchmark CHRYSLER RM was used for the Orleans Avenue Canal floodwall (Table 12).

Table 10. Geodetic and Mean Sea Level Adjustments on GNO Lakefront Benchmarks: 17th St. Canal (from IPET 2006b, II-76)

Elevation (ft) Below BM ALCO 1931 Referenced to Various Datums

NGVD29	NAVD88	LMSL	EPOCH	LMSL05-	LMSL01-	LMSL78-	NAVD88
		5.64	01-05	0.00			0.50
		5.88	83-01	-0.24	0.00		0.26
		6.04	60-78	-0.40	-0.16	0.00	0.10
	6.14		2004.65	-0.50	-0.26	-0.10	0.00
	6.59		1996	-0.95	-0.71	-0.55	-0.45
	6.57		1994	-0.93	-0.69	-0.53	-0.43
6.76			1991	-1.12	-0.88	-0.72	-0.62
6.87			1982	-1.23	-0.99	-0.83	-0.73
6.96			1986	-1.32	-1.08	-0.92	-0.82
7.38			1965	-1.74	-1.50	-1.34	-1.24

Table 11 Geodetic and Mean Sea Level Adjustments on GNO Lakefront Benchmarks: London Ave. Canal (from from IPET 2006b, II-92)

Elevation (ft) of BM P-153 1951 Referenced to Various Datums

NGVD29	NAVD88	LMSL	EPOCH	LMSL05-	LMSL01-	LMSL78-	NAVD88
		9.31	01-05	0.00			0.48
		9.54	83-01	-0.23	0.00		0.25
			60-78				
	9.79		2004.65	-0.48	-0.25		0.00
	10.21		1996	-0.90	-0.67		-0.42
	10.20		1994	-0.89	-0.66		-0.41
10.39			1991	-1.08	-0.85		-0.60
10.62			1986	-1.31	-1.08		-0.83
10.71			1982	-1.40	-1.17		-0.92
11.27			1965	-1.96	-1.73		-1.48

Table 12. Geodetic and Mean Sea Level Adjustments on GNO Lakefront Benchmarks: Orleans Ave. Canal (from IPET 2006b, II-83)

Elevation (ft) at BM CHRYSLER RM 1931 Referenced to Various Datums

NGVD29	NAVD88	LMSL	EPOCH	LMSL05-	LMSL01-	LMSL78-	NAVD88
		5.64	01-05	0.00			0.48
		5.88	83-01	-0.21	0.00		0.27
		6.04	60-78	-0.36	-0.15	0.00	0.12
	6.42		2004.65	-0.48	-0.27	-0.12	0.00
	6.85		1996	-0.91	-0.70	-0.55	-0.43
	6.83		1994	-0.89	-0.68	-0.53	-0.41
7.03			1991	-1.09	-0.88	-0.73	-0.61
7.23			1986	-1.29	-1.08	-0.93	-0.81
7.11			1982	-1.17	-0.96	-0.81	-0.69
7.69			1965	-1.75	-1.54	-1.39	-1.27

The top three rows of each set of adjustments give the local mean sea level (LMSL) determined by the NOAA CO-OPS program at this point for the 2001-05, 1983-01, and 1960-78 epochs of water level measurements. The fourth row gives the elevation in the IPET time-stamped NAVD88 adjustment (2004.65). This adjustment of NAVD88 has been employed by all investigators since Hurricane Katrina.

The fifth column headed LMSL05 (Local mean sea level 2005) in any of the benchmark histories shows mean sea level rose about 0.2 feet between the 1983-01 and the 2001-05 epochs. The total sea level change between the 1960-78 and 2001-05 epochs, where shown, is 0.4 feet. So, local mean sea level has risen relative to these benchmarks by 0.4 feet since the authorization of the GNO HPS in 1965. The offset of the current geodetic datum (NAVD88 2004.65) below LMSL05 is about 0.5 feet, as can be seen in the top row of the last column. LMSL today is about half a foot above the NAVD88 (2004.65) datum plane along the GNO lakefront.

The benchmark is fixed to a structure like a floodwall or bridge and has been sinking relative to a truly motionless datum plane represented by NGVD29, and later by NAVD88. This is indicated by the increasingly negative numbers going from top to bottom (i.e. back in time) in the four columns on the right side of each table. The numbers are slightly less negative when the older benchmark adjustments are compared to NAVD88 in the last column, than they are when compared to any of the mean sea level elevations in the three columns immediately to the left. This is because local mean sea level (LMSL) is going up while the benchmark is also sinking, so the local mean sea level and the older benchmark adjustments diverge over time (Figure 84).

Engineers at the New Orleans District used the 1965 NGVD29 datum plane as a proxy for local mean sea level, interchanging the references on design documents and drawings. In 1965, it can be seen that the elevation of mean sea level at that time (LMSL78) above the NGVD29 datum varied from 1.34 to 1.39 ft along the lakefront. That separation increases if the LMSL01 or LMSL05 values are compared to the 1965 NGVD29 elevations used for control. **The floodwalls along the London Avenue Canal were built in the early 1990s, and the crest elevation for the I-walls was specified in the design at 14.4 ft above mean sea level. But if it was constructed assuming that the 1965 NGVD29 elevation of benchmark P153 was equivalent to mean sea level, then the top elevation would be built about 1.7 feet low relative to the actual mean sea level at that time, at about 12.7 feet LMSL01 (Figure 83).**

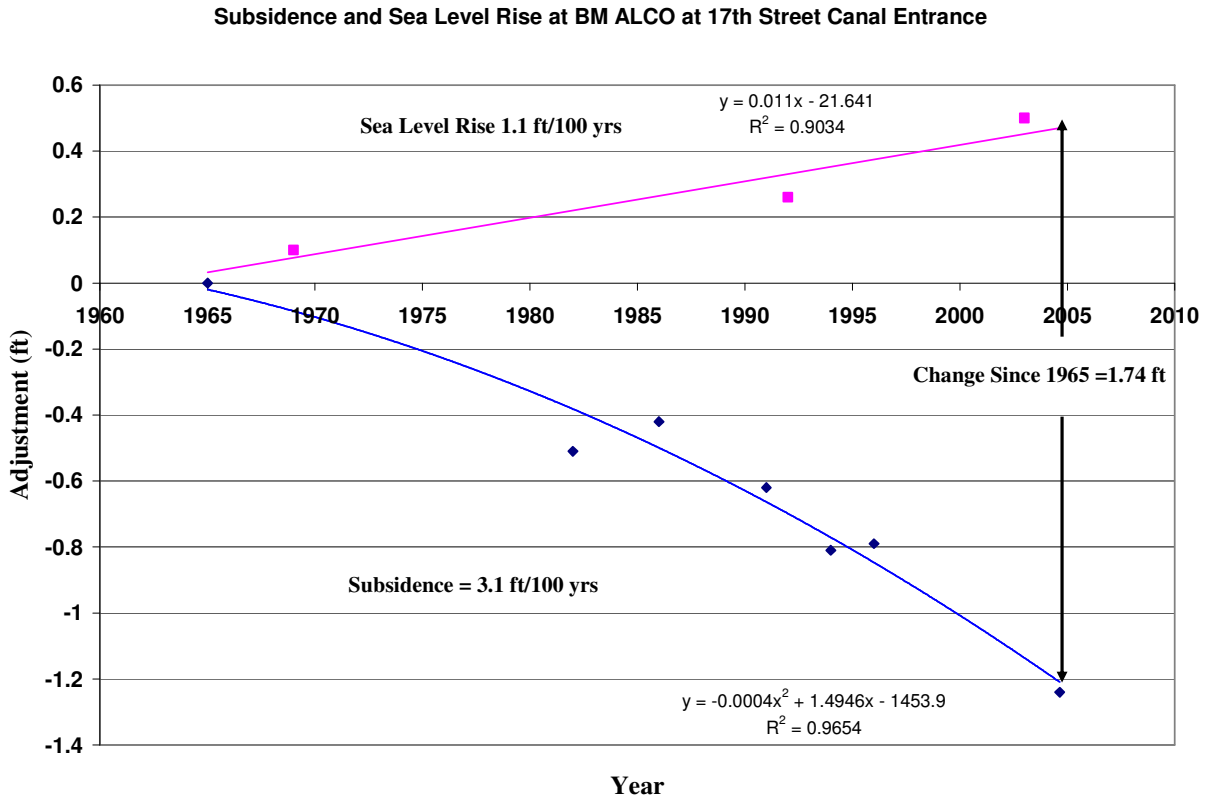


Figure 84. Trends in Sea Level and Subsidence at BM ALCO at Mouth of the 17th Street Canal from Table 6.1

Table 13 London Avenue Canal Floodwall: Design and Current Top Elevation (ft) Relative to Local Mean Sea Level (Adapted from IPET 2006b, II-94, Table 19)

Canal Section	Design El. MSL	Current Elevation NAVD88(2004.65)	Current Elevation LMSL01 and (deficit)
West Bank			
Leon Simon to R.E. Lee	14.4	13.0	12.8 (-1.6)
R.E. Lee to Filmore Ave.	14.4	12.9	12.7 (-1.7)
Filmore to Mirabeau Ave.	14.4	12.9	12.7 (-1.7)
Gentilly to Pump	14.4	12.9	12.7 (-1.7)
East Bank			
Leon Simon to R.E. Lee	14.4	12.8	12.6 (-1.8)
R.E. Lee to Filmore Ave.	14.4	12.9	12.7 (-1.7)
Filmore to Mirabeau Ave.	14.4	12.9	12.7 (-1.7)
Mirabeau to Gentilly	14.4	12.9	12.7 (-1.7)
Gentilly to Pump	14.4	13.1	12.8 (-1.6)

In 2005, a surveyor measuring the elevation of the top of the London Avenue Canal floodwall relative to the latest IPET geodetic adjustment would report a wall elevation of 12.7 feet NAVD88(2004.65). The deficiency between the design elevation and the as-built elevation, when referenced to mean sea level during the epoch they were constructed (LMSL01) is between 1.6 and 1.8 feet, as would be expected if the 1965 NGVD29 benchmark elevation was used as a proxy for mean sea level (Table 11). Results were similar for the 17th Street Canal floodwalls, but the deficiencies on the Orleans Avenue Canal were 0.2 to 0.5 ft less. It may be that a later NGVD adjustment was used to set the top elevation there.

We can attribute the current deficiencies in the floodwall elevations on the Orleans Metro drainage canals entirely to (1) the conflation of NGVD29 and LMSL (local mean sea level), and (2) use of outdated adjustments on the NGVD29 benchmarks. There is no evidence that settlement of the walls themselves played any role at these locations.

IPET points out that since surge elevations in the 17th Street and London Avenue canals at the time of initial failure (6.5 to 8.2 feet) were less than the current or pre-Katrina elevations of the floodwalls (Table 13), the lost freeboard should not have affected the breaching process that was caused by poor foundation design. On the other hand, the deficiencies on these walls -- among the most recent additions to the HPS -- establish a need to carefully reconstruct the datum guidance for other parts of the HPS were prolonged overtopping of walls and levees that were too low may have led to breaching.

Top Elevation Deficiencies due to Datums on IHNC Floodwalls

IPET has also established that here LMSL was interchanged with NGVD29 for the I-walls constructed in the early 1970s along the IHNC. An excerpt from a 1969 design drawing shows the elevation of the controlling benchmark, M152 is labeled as having an elevation of 21.81 ft MSL (Figure 85), but this value also turns out to be the 1965 NGVD29 elevation (Table 14). If the floodwall was designed to have a crest elevation of 15 feet MSL, but the 1965 NGVD29 benchmark elevation was used to control construction, the wall would have been built almost 1.7 feet below estimated local mean sea level at that time, at about 13.3 feet LMSL78 (Figure 86).

Local mean sea level relative to the IPET NAVD(2004.65) datum in the IHNC is currently estimated at +0.68 ft, almost 0.2 ft higher than along the Lake Pontchartrain shoreline, and has risen almost 0.5 ft since the IHNC I-walls were built (Table 14). The 1965 NGVD29 elevation was the latest adjustment at the time these structures were built, but the result is the same as for the Orleans Metro drainage canal floodwalls. **At the time Katrina struck, we can account for more than 2 ft of the observed**

deficiencies in IHNC floodwall elevations through (1) the conflation of NGVD29 and LMSL, and (2) lack of provision for sea level rise. Localized settlement played a lesser role in some places, as would be expected for an older structure.

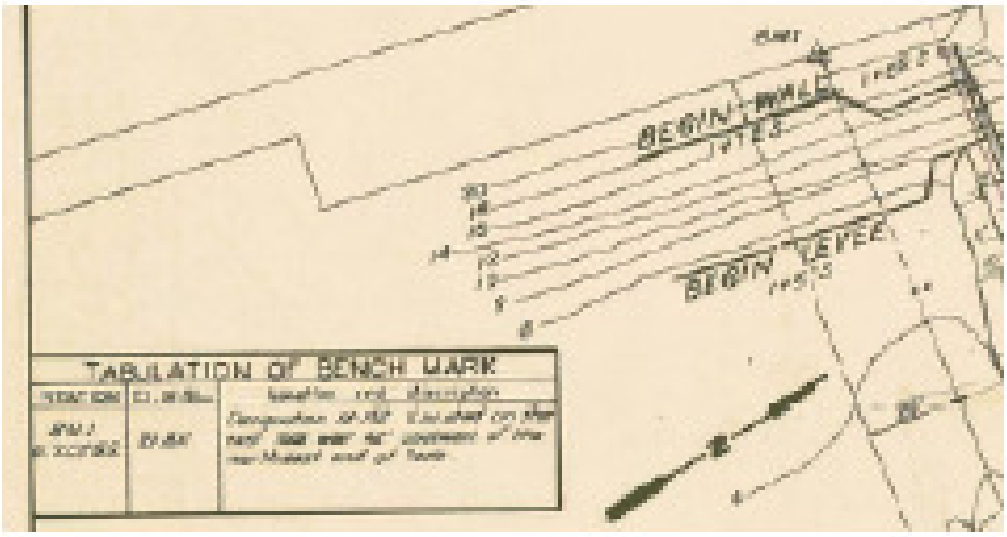


Figure 85. Portion of construction plans for IHNC east bank I-wall showing that the elevation of benchmark M 152 is given as 21.81 feet MSL, which is the 1965 NGVD29 value (IPET 2006b, Vol. II-107).

Table 14. Geodetic and Mean Sea Level Adjustments on IHNC Benchmark located on east wall of the IHNC lock (from IPET 2006b, II-109)

Elevation (ft) at BM M152 Referenced to Various Datums

NGVD29	NAVD88	LMSL	EPOCH	LMSL05-	LMSL01-	LMSL78*	NAVD88
		19.70	01-05	0.00			0.64
		20.00	83-01	-0.30	0.00		0.34
		20.16*	60-78	-0.46	-0.16	0.00	0.18
	20.34		2004.65	-0.64	-0.34	-0.18	0.00
	20.81		1996	-1.11	-0.81	-0.65	-0.47
	20.76		1994	-1.06	-0.76	-0.60	-0.42
20.96			1991	-1.26	-0.96	-0.80	-0.62
21.07			1986	-1.37	-1.07	-0.91	-0.73
21.15			1982	-1.45	-1.15	-0.99	-0.81
21.81			1965	-2.11	-1.81	-1.65	-1.47

* Estimated values for 1960-78 LMSL, no data in IPET

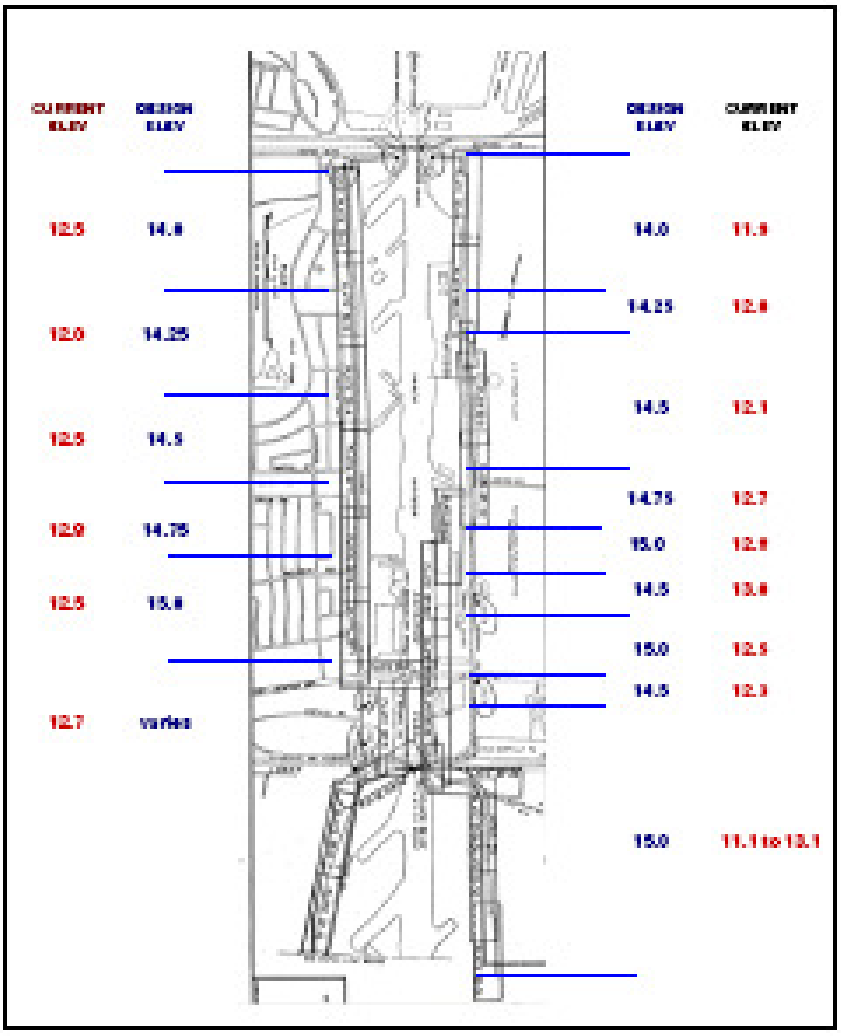


Figure 86. IPET comparison of design and current top of floodwall elevations referenced to LMSL05 for the northern reach of the IHNC. A considerable range in the deficiency is noted, from 1.5 to more than 2.0 feet. (IPET 2006b, Vol. II-113).

When the crests of undamaged floodwalls adjacent to the southern breach on the east bank of the IHNC adjacent to the Lower 9th Ward were surveyed after the storm, they had an average elevation of 12.5 feet NAVD88(2004.65), which is about 1.5 feet below the 15 feet LMSL78 level specified in the design. Top of floodwall elevations recently surveyed farther north on the IHNC by IPET vary from 1.5 to more than 3 ft below the design elevation (Figure 86).

The Katrina surge elevation measured at the IHNC lock was determined to be 14.2 ft relative to NAVD88(2004.65), and was perhaps a foot higher at the junction with the MRGO. Waves there were estimated at less than 1.5 ft and were lower elsewhere (IPET 2006b, IV-227, Fig. 163). If the floodwall crowns were built to the design specification,

they would have had an elevation of approximately 14 ft NAVD88(2004.65), or no more than a foot below the maximum surge elevation anywhere in the IHNC.
Given the variation in floodwall crown elevations that IPET documented farther north, it is possible that the floodwall crest elevation at the point where overtopping initiated the south breach into the Lower 9th Ward could have been a foot lower than the 12.5 ft found in surviving adjacent segments.

IPET and Team Louisiana have established through geotechnical analysis that the I-walls in the south breach area on the east side of the IHNC adjacent to the Lower 9th Ward were, unlike their cousins on the drainage canals, sufficiently robust to withstand surge up to the full height of the wall. IPET has also concluded that the larger, southern breach into the Lower 9th Ward was caused by overtopping and erosive removal of the protected side of the earthen levee foundation. ILIT has postulated that underseepage may also have played an important role in the north breach.

Regardless of which mechanism contributed most to the failure, it is hard to overestimate the significance to the breaching process of constructing the wall almost 2 feet low. From the surge hydrograph, we can determine that overtopping would have been initiated about 3 hours later if the wall was built to a true LMSL78 value. **Given that many miles of low IHNC I-walls withstood the surge even though they were overtopped for a prolonged period, it is likely that the large catastrophic southern breach in the Lower 9th Ward, as well as the smaller to the north, may not have occurred if the proper crown elevation had been specified for construction.**

Corps Policy on Datums and Subsidence
The U.S. Coast & Geodetic Survey, later the NGS, notified the USACE New Orleans District (NOD) of subsiding benchmarks in the GNO in 1958 (IPET 2006b, II-132). It appears clear now that the NOD did not use this information when developing initial plans and cost estimates for the 100-year Lake Pontchartrain & Vicinity Hurricane Protection Project prior to the 1965 authorization. This blindness to the effects of continuing subsidence on the effective level of flood protection continued at least through the 1990s (Figure 87).

In 1973, the NGS renamed the Sea Level Datum of 1929 (SLD29) the National Geodetic Vertical Datum of 1929 (NGVD29) to "avoid the costly errors that may result through failure to consider local sea level when engineering projects are undertaken." A Corps directive issued October 1978 (ETL 1110-1-97) was broadcast to all districts five years after the NGS action. It is possible that the older name may have been at the root of continuing confusion in the NOD.

After discussions over the previous year with the NGS, the USACE Lower Mississippi Valley Division (LMVD) and with local sponsors, NOD wrote a memo in August 1985 asking the LMVD for approval of a proposed policy for implementing, or not implementing, new NGS adjustments to benchmarks. NOD saw a serious problem in transitioning to up-to-date benchmark elevations. Previously constructed flood protection projects would *offer* a lower level of protection than those constructed later. NOD noted that compliance to increase grade could require demolition and reconstruction of completed or partially completed sections. The relevant portions of the proposed policy are given below.

a. NOD agreed to set water level gages to the new datums so that the public is not "confused."
b. NOD would make no change in completed projects in response to the new information.
c. NOD would continue to construct Mississippi River main stem improvements using best pre-1983 information available as critical elevations are driven by discharge and it is assumed that subsidence affects both bed and banks equally. NOD acknowledged that this could cause some problems in tidal reaches.
d. NOD agreed to use latest benchmark data published at the time construction started.
e. NOD proposed that dredging projects be allowed to use pre-1983 control because they would otherwise have to dredge everywhere about 1 ft deeper than at present.

The most important provision regarding the GNO HPS follows.

f. Hurricane Protection projects which are partially complete will use the NGS benchmarks current at the time of construction of the first increment of the project. To shift to the later NGS data without altering the heights of previously constructed portions would make "fuseplugs" of those portions and thus impose a gratuitous servitude on the lands and facilities they protect. And altering previously constructed works would not be practicable.

Significant portions of the 1965 authorized GNO HPS project had not yet been designed, much less constructed by 1985, particularly the I-wall capping along the east bank of the Duncan Canal on the west side of the Jefferson Parish East Bank and the Orleans Metro drainage canals. **Yet the District was requesting that the LMVD approve a policy that would specify use of a superseded datum without any analysis**

of the effect of this policy on the capacity of the levees and floodwalls to hold back SPH surge and waves. NOD, knowingly, was reducing the crown elevations and therefore the safety of New Orleans residents. NOD also proposed that they not provide funding to the NGS to study subsidence in the GNO area, following a similar initiative in the Houston-Galveston area, suggesting either a lack of interest or concern about what such a study might reveal.

NOD was effectively asking LMVD for permission to turn the clock back two decades to 1965. Nevertheless, this policy was approved by LMVD the following month with the caveat that "consideration should be given to reanalyzing and modifying (if needed) hurricane protection work in high density urban areas where the datum changes will drastically reduce the level of protection."

This advice was never followed by NOD or enforced by LMVD. There is no mention of local sea level, and the 1985 policy would only lock in the datum deficit. It appeared to ensure that no additional provisions for subsidence or sea level rise would ever be considered for the GNO HPS. Indeed, IPET reports (IPET 2006b, Vol. II-78):

> "In reviewing the DMs and related documents, it was not clear how projected subsidence rates were applied in structural elevation design, if at all. Subsidence was apparently not factored into the design freeboard allowance. Most subsidence estimates in the DMs were small – e.g., less than 1 ft/century – almost negligible."

As we have seen, typical subsidence values for benchmarks in the GNO area are on the order of 3 ft/century at the GNO lakefront, and higher, more than 4 ft/century, at the IHNC Lock (Figure 87). The trends have been consistent since at least 1952 and should have been recognized in 1965 when the GNO HPS was authorized. The potential for subsidence of areas under forced drainage has been documented in the Mississippi River delta since the early 1900s (Harrison and Kollmorgen 1947).

IPET reports that in 1987, the NOD circulated an internal memorandum to include an extra foot of freeboard on levees under design for the West Bank of the GNO to accommodate "subsidence rates in some parts of the project area of as much as 2.0 to 2.5 feet per century" (IPET 2006b, II-135). There is no mention of the East Bank HPS.

On April 1, 1993, USACE headquarters (HQUSACE) issued a nationwide policy directive for referencing coastal navigation projects to the (NOAA) mean lower low water datum (ETL 1110-2-349 and EM 1110-1-1005) based on language in the Water Resources and Development Act of 1992 (WRDA 92). IPET reports that this policy was

not implemented by the NOD "presumably under the interpretation that WRDA 92 could not supersede the original project authorization documents containing the older Mean Low Gulf (MLG) datum reference" (IPET 2006b, II-22). This rationale may also have been at the root of reluctance to apply post 1965 LMSL and datum adjustments for the GNO HPS.

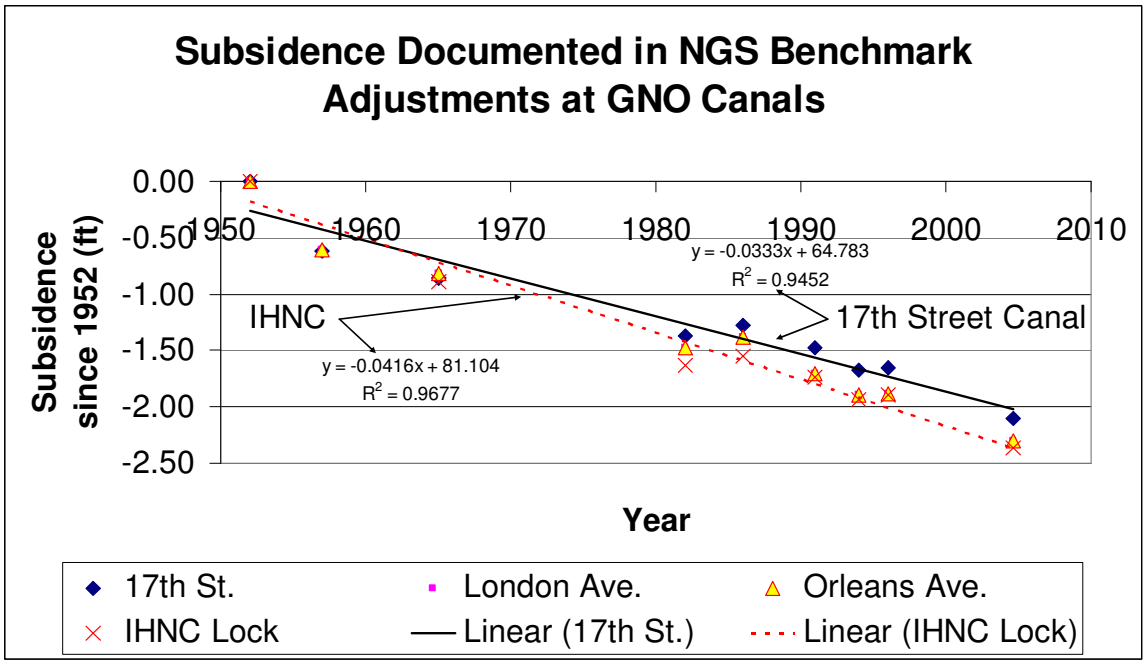


Figure 87. Subsidence documented in NGS benchmark adjustments at GNO canals relative to NAVD (2004.65). The linear trend for the 17th St. Canal is typical of the lakefront at 3.3 ft/century. Subsidence is greater, 4.2 ft/century, at the IHNC Lock

In August, 1993, the Orleans Levee District requested NOD to address the problem that they had discovered in construction plans for the London Avenue Canal. The Levee District pointed out that "flood protection is higher for the Orleans Canal than that for the London Canal. This appears to be an intolerable situation. Please adjust as required to provide maximum protection for both canals." IPET (2006b) also found this letter from the levee board to NOD. IPET had access to over a hundred boxes of data and records from NOD. We certainly did not. However, IPET states that no response is noted to the levee board letter. Louisiana Department of Transportation and Development (LDOTD) officials have made similar requests on numerous HPS structures over the years, and reported considerable frustration with the lack of constructive response from NOD.

In 1994, HQUSACE issued new technical guidance (ETL 1110-1-152) to all districts mandating conversion to NAVD88, the national system in use since 1992. Specific

guidance was provided on interactions with the National Flood Insurance Program regarding use of local sea level, and the need to address subsidence in coastal areas (IPET 2006b, II-135).

In October, 2000, NOD issued a Policy Letter asking LMVD to approve a retraction of some of the most egregious parts of the previously approved 1985 memo based on its **"untenability" given the extent of complaints from local sponsors.** This policy letter also proposes a plan for transitioning to NAVD88 six years after 1994 HQUSACE directive ETL 1110-1-152. LMVD concurred with the NOD proposal in January, 2001, but also asked again that NOD begin an evaluation of completed projects to determine whether or not modifications were necessary to achieve authorized levels of protection. LMVD asked for semi-annual status reports, but no documents of this type have yet been produced or made public. The NOD did not effectively implement NAVD88 until two years later, in 2002 (IPET 2006b, II-22).

As part of its post-Katrina investigation, IPET interviewed NOD managers to try to understand what appeared to be a nearly two decades of delay in addressing datum and subsidence issues affecting the GNO HPS, once they were revealed by other agencies and local sponsors. It does not appear that the NOD ever would have found these problems otherwise. Apparently, NOD planned to reassess the actual level of protection only after a lengthy multi-agency effort to establish vertical control was completed. This effort was not complete when Katrina struck. NOD managers stated that "funding levels were insufficient to complete construction at existing holes in the system that were crucial to flood protection…hence levees were below grade in quite a few areas" (IPET 2006b, II-22). The question we ask is "Why was the public not informed?"

It appears that NOD was consumed by an imperative to complete what they interpreted as the originally authorized HPS within whatever budget was available, regardless of the effect of datum deficiencies or subsidence on the level of protection that was actually being provided. The levees and floodwalls were built lower than they should have been because of an unshakable erroneous assumption that the NGVD29 datum plane was equivalent to LMSL. This problem is understandable if not excusable, but it was further amplified by a policy decisions to explicitly ignore datum issues, subsidence and sea level rise in the construction of the GNO HPS.

Effect of Corps policy on Variances in Crown Elevations and Flooding

The 1959 SPH and early 1960s oceanographic analysis set minimum crown elevation heights necessary to prevent overtopping by surge and waves. The designers added some freeboard in some cases for various reasons. By the time many of the elements of

the GNO HPS were constructed 20 to 30 years later, long after the original SPH analysis was known to be obsolete, the design level of protection on floodwalls was reduced by 1 to 2 feet because of the datum issues discussed above. Because no provisions were made to accommodate subsidence rates that were known at the time of authorization, or sea level changes that became apparent later, the 100-year project was deficient from the beginning, and would deliver an ever lessening level of protection through its design life.

Earthen levees were generally constructed in stages over many years to allow for consolidation and to improve strength between lifts. Each lift would be raised above whatever target elevation was specified to allow for subsequent settlement. It is more difficult to clearly separate the effects of datum confusion from subsidence and settlement of the levee. In other places, differences in crown level may have been a result of construction at different times. A classic case is presented by the transition between the Citrus Lakefront Levee and the New Orleans East Lakefront Levee that meet at the northern terminus of Paris Road in New Orleans East (Figure 88 a and b). The New Orleans East Lakefront Levee has a design level of protection of 18 ft, and protects the undeveloped marsh of the Bayou Sauvage National Wildlife Refuge, while the Citrus Lakefront Levee protects the Little Woods residential area and has a design elevation of 14.5 ft, but may have some foreshore protection against waves (Figure 88b).

Figure 88a. Adjacent federal levees with a 4-foot offset facing Lake Pontchartrain in New Orleans East. The Citrus Lakefront levee on the left has a design elevation of 14.5 feet and protects a densely populated suburb. It experienced wave overwash. The New Orleans East Lakefront on the right has a design elevation of 18 feet and protects the undeveloped marshes of the Bayou Sauvage National Wildlife Refuge. It was not overtopped. The road in the foreground is on the crest of a much lower local levee that was extensively overtopped.

Figure 88b. Same levees as in Figure 88a, except view is looking south from Lake Pontchartrain, August 2006.

We analyzed the pre-Katrina level of deficiency relative to current sea level (LMSL05) for each of 22 HPS named levee/floodwall reaches (with GDMs) using 1999 LIDAR data, limited ground truth surveys and post-storm survey data scattered throughout the final IPET report (Table 15). The most widespread cause of levee and floodwall failure was overtopping and erosion. Here, we summarize the relationship between crown deficiencies relative to the design elevation and the severity of breaching observed during Katrina.

We found that of the 22 HPS elements analyzed, 18 had deficiencies of 2 feet or more from the design level for the protective structure when assessed relative to current sea level (LMSL05). The exceptions were the Jefferson Parish Lakefront, New Orleans East Lakefront and New Orleans East South Point to Highway 90 levees that were exposed to surge and waves from Lake Pontchartrain, and performed well. The Orleans Lakefront and Citrus Lakefront Levees were more than 2 ft deficient in places, but they did not experience significant overtopping and were not breached. Floodwalls along the Orleans Metro drainage canals and the Jefferson/St. Charles Return Levee along the Duncan Canal all were deficient by 2 ft or more, but they were not overtopped, and the breaches that did occur were a result of poor foundation design. Seven levee or

floodwall reaches had deficiencies of 4 feet or more. Except for the uncapped Orleans Avenue Canal levee which overtopped but did not breach, all of the most deficient levee reaches were affected by the Lake Borgne funnel and were exposed to the highest surge and largest waves experienced in the GNO area.

Table 15. Design Elevation of Protection, Pre-Katrina Elevation, Surge Level and Damage for GNO HPS Elements exposed to Lake Pontchartrain and Lake Borgne (ft, LMSL05)

Damage Code: NO – No Overtopping; LO – Levee Overtopping; WO – Wall Overtopping; NB – No Breach; B – Levee Breach; WF - Wall Failure; TF – Transition Failure

Levee/Floodwall Reach	*Katrina Surge (Damage Code)*	*Design Elevation*	*Observed Elevation*	*Deficiency*
	East Bank Jefferson Basin – Lake Pontchartrain System			
Jefferson/St. Charles Return	9.5 (NO)	***14.5***	***12.0-13.0***	***1.0-2.0***
Jefferson Lakefront	9.5-11.0 (NO)	***16.0***	***15.0-16.5***	***0.0-1.5***
	Orleans Metro Basin – Lake Pontchartrain System			
Orleans Lakefront	11.0-12.5 (NO)	***17.0-20.0***	***13.5-18.0***	***0.5-4.0***
17th St. Canal	11.3 (WFNO)	***14.0-15.0***	***12.2-13.0***	***1.5-2.0***
Orleans Ave. Canal	11.3 (LONB)	***14.1-14.7***	***9.1-14.3***	***1.5-5.6***
B. St. John Closure	11.5 (NO)	***18.5***	***15.5-17.5***	***1.0-4.0***
London Ave. Canal	11.5-12.0 (WFNO)	***14.0-14.4***	***12.5-13.5***	***1.0-2.5***
	Inner Harbor Navigation Canal – Lake Pontchartrain and Lake Borgne Systems			
IHNC North of MRGO	12.0-15.5 (WOTF)	***14.0-15.0***	***11.1-13.0***	***1.5-3.0***
IHNC South of MRGO	14.8-15.5 (WOF)	***14.5-15.0***	***11.5-13.0***	***1.5-3.0***
	New Orleans East – Lake Pontchartrain and Lake Borgne Systems			
Citrus Lakefront	12.7-13.4(WONB)	***14.5***	***11.0-13.0***	***1.5-2.5***
NO East Lakefront	13.4-15.0 (NO)	***18.0-20.0***	***17.5-19.5***	***0.0-1.5***
NO East S. Pt to Hwy 90	15.0-16.6 (LONB)	***13.5-14.5***	***12.0-15.0***	***0.0-1.5***
NO East Hwy 90 to GIWW	15.0-16.6(LOBTF)	***15.5-17.5***	***13.0-18.5***	***0.0-2.5***
NO East Back Levee	16.0-18.0(LOBTF)	***19.0-23.0***	***14.5-21.5***	***2.0-4.0***
Citrus Back Levee,E. Paris Rd	16.5-17.7 (LOBTF/WCF)	***18.0***	***16.5-17.0***	***1.0-1.5***
Citrus Back Levee,W.Paris Rd.	14.8-16.5 (LONBWF)	***15.0***	***12.5-14.5***	***0.5-2.5***
	St. Bernard -- Lake Borgne System			
Chalmette West of Paris Rd	14.8-16.5 (LONB)	***15.0***	***11.0-16.0***	***0.0-4.0***
Chalmette Paris Rd to B. Lawler	16.5-18.5 (LOBWFTF)	***17.5***	***15.0-18.0***	***0.0-2.5***
Chalmette B. Lawler to Violet	17.5-18.5 (LOBWFTF)	***17.5***	***11.5-18.0***	***0.0-6.0***
Chalmette Ext. Dupre to Verret	16.5-18.5 (LOB)	***17.5***	***14.5-17.5***	***0.0-3.0***
Chalmette Ext. Verret to Toca	14.0-16.5 (LONB)	***16.5***	***14.5-17.0***	***0.0-2.0***
Chalmette Ext. Toca Caernarvon	11.0-14.0 (LONB)	***16.5***	***12.0-15.0***	***1.0-4.5***

All of the levee and floodwall reaches exposed to the Lake Borgne surge system, including those along the IHNC, experienced overtopping. If the IHNC floodwalls had been at the design elevation, they should have experienced less than 1 ft of overtopping

at peak surge, rather than 1.5 to 3.0 ft that occurred. This would have prevented some of the breaches and transition failures, though at least some would still have occurred at surge levels well below the wall tops as a result either of poor foundation or transition designs.

The 8.4 mile long New Orleans East Levee protecting the east side of the Bayou Sauvage National Wildlife Refuge should not have been overtopped at all, although the overwash there during Katrina appears to have been relatively minor and did not result in breaches (Figure 39). There was one wall transition failure. The sandy New Orleans East Back Levee along the GIWW would probably not have been resilient even if it was up to grade, rather than 4 ft low in places. It was overtopped by 2 feet of surge topped by substantial waves, and was constructed of a poor quality sandy hydraulic fill. Over 4 miles of levee and integrated floodwalls breached catastrophically as the surge and waves rose 4 to 5 ft over the crown elevation.

The Citrus Back Levee (along GIWW) east of Paris Road (Figure 39) was less than 2 ft below the 18 ft design level, and was constructed of resistant clay materials. It was overtopped primarily by waves (Figure 42 a.) but was not breached, and proved to be quite resilient. The design level of protection of the Citrus Back Levee to the west of Paris Road was 3 ft lower, and was perhaps as low as 12.5 ft in places (Table 15). It experienced a surge up to 16.5 ft with some unanticipated waves. Here the levee held up well under 4 ft of overtopping, but I-walls integrated into the levee failed as a result of back scour (Figure 41).

South of the MRGO/GIWW channel that forms the throat of the funnel, the Chalmette Levee west of Paris Road (Figure 44) was up to 4 ft deficient in spots, but, like the Citrus Back Levee on the north side of the channel, it held up well under 4 to 6 ft of overtopping. The extent of the overtopping was exacerbated by the deficiencies, and hastened the filling of the St. Bernard wetland buffer to the south, but this 7 mile levee reach did not breach.

The 12 miles of the Chalmette levee that follows the south bank of the MRGO were deficient from the 17.5 ft design elevation by up to 5 ft (Table 15). This levee was also constructed primarily of hydraulic fill obtained from the MRGO channel, much of which was sand, silt and shell. It is essentially a sea dike that experienced the most severe wave attack of any levee reach in the GNO HPS (Figure 44). If it was up to grade and had been built to survive the wave attack with the crown intact, it would eventually have been overtopped by 2 to 3 ft of surge and waves.

Because of grade deficiencies and the poor materials used in its construction, however, there is little doubt that most of this levee was washed away before the surge ever reached the design level of protection (Figure 89a). Failures of transitions, sheetpile walls and bayou closures also occurred in this 12 mile levee run, but most damage was done by wave-induced overtopping and erosion, as was discussed in the previous chapter.

Overtopping of a resilient MRGO levee constructed to the design level (MSL) would have delivered only about half of the volume necessary to fill the 32,000 acre wetland behind the levee, and initiate overtopping of the 40 Arpent back levee. Had this levee failed at a later stage in the surge hydrograph, or failed less completely, it is possible that the storage capacity of the 32,000 acre wetland buffer would have absorbed the discharge across the levee alignment for long enough to save Chalmette and the rest of St. Bernard from the disastrous flooding that occurred across the local 40 Arpent levee.

The last levee reach to be considered is the 11 miles of the Chalmette Extension that turns south from the MRGO and to the west to rejoin the main east bank levee of the Mississippi River (Figure 44). This levee was fronted by wetlands and did not experience the kind of wave attack that affected the MRGO levee (Figure 89b). It was also built more robustly with a river sand core covered by a substantial thickness of hauled clay. This levee experienced overtopping in places where it was up to 4.5 ft deficient in elevation relative to the design, but there was no breaching and very little damage overall. The volume of water contributed by overtopping of this reach was negligible compared to what came across the failed MRGO alignment.

In summary, the design elevations of most levees, floodwalls and structures affected by the Lake Borgne funnel were less than required to prevent overtopping by Katrina's surge and waves. This could have been prevented only if the pre-1965 hurricane oceanography had been updated to incorporate the 20 percent increase in maximum wind speed specified in 1973 and expanded in 1979 SPH (see Figure 81a). But most levee and floodwall crown levels were deficient by more than 2 ft below the already inadequate design level, at least in part because (1) of the confusion of NGVD29 with local mean sea level, and (2) a lack of accommodation for the subsidence rates of 3 to 4

Figures 89a,b. Difference in damage experienced by two Chalmette levees that experienced overtopping. The view on left is of the back side of the MRGO levee southeast from the Bayou Dupre structure. Sand and shell hydraulic fill materials are exposed. The view on right is west on the unprotected face of the Caernarvon levee on the south side of the St. Bernard basin. It has a sand core but was covered with a thick clay cap. This levee faced a broad expanse of marsh that protected it from wave damage.

ft/century known since at least the 1950s to affect the GNO area. These deficiencies did not cause the foundation failures that doomed the Orleans Metro, but clearly had an effect in the failure into the Lower 9th Ward. These deficiencies undoubtedly contributed significantly to the early onset and depth of flooding in New Orleans East and St. Bernard.

Summary Findings Regarding the Design Level of Protection

Forensics Question 2

Were the levels of protection, or crown elevations, specified in designs for HPS elements sufficient to resist overtopping by surge and waves associated with the 100 year Standard Project Hurricane?

Answer 2

No. Floodwall and levee crown elevations were built 1 to 2 ft low because of an erroneous assumption at USACE New Orleans District (NOD) that an elevation of zero referenced to the National Vertical Geodetic Datum of 1929 (NGVD29) was equal to -- and interchangeable with -- local mean sea level (LMSL). LMSL was the relevant datum for superimposition of hurricane surge and wave height. In 1965, zero NGVD29 was between 1.3 and 1.6 feet below LMSL, and floodwalls and levee crowns were constructed lower by this margin. This mistake was locked in for continuing HPS construction when the NOD adopted a policy in 1985, with the approval of the USACE Lower Mississippi Valley Division (LMVD), to explicitly use the outdated 1965

NGVD29 adjustment for elevation control, and to ignore subsidence and sea level rise. As a result, no provision was made to account for the 3 to 4 ft/century subsidence rates characteristic of the GNO area even though this rate was known at the time of authorization.

No consideration was made for the continuing coastal land loss in determining crown elevators even though the NOD had determined 2.7 miles and wetlands reduce the surge by 1 foot.

Crown elevation deficiencies ranging up to 6 feet at the time Katrina struck resulted in prolonged overtopping of floodwalls and levees along the IHNC and to the east in the Lake Borgne funnel that otherwise would have been overtopped only briefly. Prolonged overtopping led to catastrophic breaches into the Lower 9th Ward on the east and into Orleans Metro on the west, and contributed to the early failures of levees along the GIWW and MRGO. Early failure of the MRGO levee allowed the 32,000 acre wetland buffer to fill and overtop the 40 Arpent back levee while the surge was still rising, and resulted in catastrophic flooding in St. Bernard to an elevation of 11 ft (NAVD88).

CHAPTER SIX

DID THE USACE FOLLOW EXISTING GUIDANCE FOR CONSTRUCTION OF LEVEES AND FLOODWALLS?

Forensics Question 3: Did the USACE follow existing engineering practice and USACE guidance for construction of levees and floodwalls? Should issues about levee materials and floodwall designs have been detected and corrected by engineers equipped with the tools available at the time of construction?

Answer: No to the first question, and yes to the second. Weak soil strengths or potential for underseepage were evident in strata tested for the USACE during the early 1980s that underlie Orleans Metro drainage (outfall) canal floodwall levees that failed. The potential consequences of these layers on levee stability were known to practicing engineers at the time, but were missed or ignored because of inappropriate averaging of soil strengths on long levee reaches and across layers. Consolidation of soils beneath the I-wall levees on the 17th Street Canal should have increased soil strengths over time, but borings and soundings conducted since Katrina show that very soft clays in the 17th Street Canal failure zone still have strengths less than values assigned in 1981. Where Division-level reviewers identified potential problems, they were rebuffed by District personnel citing "professional judgment."

The New Orleans District (NOD) USACE failed to conduct appropriate analyses of the potential for seepage to compromise levee and floodwall stability where shallow sand deposits occurred beneath the levee, as on the London Avenue Canal. Design memoranda indicate reliance upon the Lane's Weighted Average Creep method for underseepage analysis. This method was recognized in the profession at the time to be inappropriate for final design in a critical life-support structure. The presence of layered sands and clays should have led to analysis using more rigorous flow net and finite-element techniques, in widespread use at the time and specified in the governing USACE engineering manual for Design and Construction of Levees (EM 1110-2-1913, 1978 ed.). There is no evidence that rigorous analysis of uplift pressures was undertaken.

Idealized design templates were applied to levee and floodwall reaches that were too long, without adjustment for variable subsoil conditions or for variations in elevations on the protected side. IPET says that such a mistake caused the levee supporting the I-wall levee to be built too low in the vicinity of the north breach into the Lower 9th Ward.

Sheetpile supported I-walls were installed or substituted for more robust T-Walls on levees with cross-sections too small to prevent underseepage in some cases, or to provide sufficient resistance when fully loaded, no matter what the sheetpile length.

The NOD USACE did not follow standard engineering practice or Corps guidance when evaluating whether to protect (armor) earthen levees or sea dikes from erosion caused by waves in the funnel area east of the city.

Levees were not designed to withstand general overtopping, as was amply demonstrated in Katrina, even though they were expected to experience overtopping by the 14% of waves greater than the significant wave provided in the oceanographic analysis.

Many miles of the Chalmette and New Orleans East Levees were constructed of shell-rich sands derived from excavation of the adjacent GIWW and MRGO channels, rather than the clay soils specified for levees protecting urban areas (EM 1110-2-1913, 1978 ed.).

Engineering Analyses of Levee and Floodwall Failures

We have focused to this point on the oceanographic and datum issues that, in combination, were used by the USACE to set the design elevation of protection for levees and floodwalls throughout the GNO HPS. Levees and floodwalls in many parts of the HPS were overtopped either because they were built too low, or because the surge and waves associated with the SPH were underestimated.

No flood protection structure constructed as part of the GNO HPS was designed to survive overtopping. None were explicitly equipped with back side scour protection although the T-wall type structures used in some flood walls inherently had more overtopping protection because of the geometry. This may, in hindsight, seem cavalier given the relatively low cost of providing this type of resilience, but it is apparently the case with most flood protection structures built in the United States. No USACE or FEMA design manuals currently call for overtopping protection, though it is likely that they will in the future. Rather, they mandate a level of protection sufficient to resist overtopping by a rare event of a given return frequency. If a more severe event occurs – as it certainly will at some point -- and the structure is breached or otherwise damaged, this is not considered failure, but the expected consequence of exposing the structure to forces beyond the design. Indeed, it could be stated alternatively that the vast majority of structures of the GNO HPS were designed to fail upon overtopping, however it occurred.

If all of the levee and floodwall failures that accompanied Hurricane Katrina were accompanied by overtopping, few would regard the destruction of New Orleans as anything but another sad reminder of the destructive and unavoidable forces of nature. But this was not the case. Many reaches of floodwalls and levees were damaged or destroyed because they were overtopped; but others failed that were never in danger of overtopping, or failed before they could be overtopped. Two hundred-twenty miles of the 350 miles of protection was damaged or destroyed and had to be subsequently repaired or replaced. These failures exposed parts of the city to flooding that would otherwise have been spared. Because these unacceptable breaches have been carefully documented and reconstructed by the IPET, ILIT, and Team Louisiana, **the flooding of New Orleans is regarded now not solely as a natural disaster, but also as a catastrophic engineering failure – a man-made catastrophe.** We address first the floodwall failures that all agree were caused by design defects, where there clearly was no overtopping. We then move on to some in which overtopping played some role, whether or not it was decisive.

Design Considerations for the Orleans Metro Outfall Canals

The parallel floodwall systems constructed along the three Orleans Metro outfall canals were not overtopped during Katrina. They have attracted a large portion of the attention of the post-Katrina investigations for four important reasons. First, it was known within the first weeks after the storm that they had failed at canal water levels that were clearly within the design safe operating range, and so could not easily be attributed to extraordinary, unanticipated forces. Second, the three drainage canal breaches produced flooding that could not be stopped for days. This led to the slow motion inundation of the commercial and residential heart of an American city. Third, because the canals were integral to the pumped drainage system, the breaches made it impossible to clear the city of floodwaters for weeks after the storm, further adding to the damage and cost of recovery. Finally, the three drainage canal floodwalls, as well as the St. Charles/Jefferson Parish Return (Duncan) Canal floodwalls, were among the most recent additions to the HPS, having been constructed in the 1990s, with ancillary elements still incomplete at the time Katrina struck. So, it could not be argued that they had been conceived in the misty past when much less was known about underlying soil conditions or how to design flood protection structures. These were modern structures designed to protect a high-value, heavily populated urban area by what was thought to be the world's premier civil engineering organization.

Parallel Floodwalls or Lakefront Gates

All of the general design memoranda (GDMs) for raising the level of hurricane protection along the Orleans Metro drainage canals provide two alternatives to achieve the same ends. The approach recommended by the USACE to provide "1 in 300 year"

protection under the "high-level" plan adopted after 1985 was to construct a "fronting protection structure" near the lakefront with specialized "butterfly gates" that would prevent the surge from entering the canals, while permitting pumping for drainage purposes. Alternatively, the USACE states in the 1988 GDM 19 for the Orleans Avenue Canal; in the 1989 GDM 19A for the London Avenue Canal; and in the 1990 GDM 20 for the 17th Street Canal that the same result can be achieved by a "parallel protection plan" requiring upgrading the height of the existing floodwalls along both sides of the canals, raising or floodproofing bridges, and providing flood closure gates at the interior pumping stations.

The USACE states in all cases that the "fronting protection plan is the most cost-effective way to provide hurricane protection; can be designed to fully accommodate existing and/or future interior drainage; and will be the least disruptive (from the stand point of construction) to protect the developed areas behind the levees." Each of these GDMs also notes that the local sponsors responsible for sharing the costs, namely the Orleans Levee Board (OLB) and the Sewerage and Water Board of New Orleans (SWBNO), have "gone on record in support of the parallel protection plan."

It is easy to understand why. First, the SWBNO did not have confidence in the untested self-actuating gate system proposed by the USACE to close the canal whenever lake level exceeded the level in the canal. This would require pumping for drainage to cease at times when the city was likely to be experiencing heavy rainfall, and perhaps would limit the potential to increase discharge in the future. Additionally, during initial gate model tests attended by board members at the Waterways Experiment Station (WES) in Vicksburg, the gates failed to work. The USACE estimated in 1988 that the fronting protection plan would cost only $15 million for the London Avenue Canal, while the parallel protection system would cost $44 million. The divergence in these estimates was similar for the other drainage canals. The USACE provided its own technical rationale for why it was promoting the fronting protection plan, and why the SWBNO was against it. The following statement is taken from GDM 19A for the London Avenue Canal, but similar words can be found in each of the Orleans Metro drainage canal GDMs.

> "Fronting protection which is designed to accommodate interior drainage, fully meets the mandate of the Project Authorization. With the line of hurricane protection established at or near the lakefront, the levees on the protected side of the structure are considered to be interior drainage features. Any existing limitations which the interior drainage system currently has will not be affected by the construction of the proposed fronting protection. The limitations referred to here concern the capability

> of the pumping stations… to pump against high lake stages… and inadequate freeboard of the existing lateral levees. Also, the SWBNO… objective is a normal lake stage and not for an SPH event which has a recurrence interval of about once in 300 years. The management of storm drainage is entirely SWBNO's responsibility. Consequently, the focus of the plan formulation process was centered on alternatives which appear cost-effective from a hurricane protection standpoint while offering optimum physical conditions for an efficient operation of the existing pumping station during hurricane events."

At the time the Orleans Metro drainage canal GDMs were written, it was already known that the local sponsors would not support the USACE recommended plan. GDM 19A reports that "development of the (parallel protection) alternative became more desirable when SWBNO expressed its strong opposition to any plan which calls for establishing a pre-agreed set of conditions for gate closure."

Now that Hurricane Katrina has flooded New Orleans through the drainage outfall canals, it would seem that local opposition to the lakefront gates was shortsighted. Indeed, less adaptive emergency gates are currently being built, and at some point they will either be replaced with gates similar to what was originally proposed, or full pumping capacity will be moved out to the lakefront. There are two weaknesses to the "I told you so" argument, however, that need some explanation. First, the 1959 SPH was being presented as a far less likely event (1 in 300 years) than it actually is, as has been discussed earlier. Local agencies had to balance that remote likelihood against the far more frequent damage caused by interior flooding events, and the chance that the untested gates proposed would compound these problems on an every day basis. Second, the local sponsors were led to believe that the parallel protection plan was technically equivalent, if more costly. They believed they had a right to pursue a more costly alternative if it would accomplish other purposes in addition to the hurricane protection objective. They believed the USACE, in recommending the less costly fronting protection plan, was simply establishing a limit to the portion that the federal government would be willing to pay.

When the local sponsors indicated that they wished to pursue the parallel protection plan, it appears that the USACE felt that it was absolved, to some degree, from some responsibility to assure the equivalence and effectiveness of the protection system. The USACE was in a better position than the local sponsors to insist that the two approaches were not, in fact, technically equivalent. They were well aware of the foundation soils problems that had bedeviled them on every aspect of the design and construction of the GNO HPS. Engineers under contract to the Orleans Levee Board set to work to try to

reduce the cost of the parallel system. As we will show, step-by-step, the USACE acquiesced in a series of design changes that ended up ensuring that the floodwalls constructed did not, in fact, provide an equivalent level of hurricane protection to that proposed in the fronting plan, even as the NOD cut construction costs.

At this point, the mechanisms of the three drainage canal floodwall breaches are understood in considerable detail as a consequence of the IPET, ILIT and Team Louisiana post-Katrina investigations. Team Louisiana engineers additionally contributed to that understanding by conducting cone penetrometer (CPT) and seismic tests in the vicinity of the breaches, and providing these results to other teams (See Appendix 9).

The IPET and ILIT have reconstructed the Orleans Metro floodwall failures in hundreds of pages of results from finite-element computer simulations, and testing in centrifuges and other small-scale physical models. These are extremely valuable contributions. From a forensics standpoint, however, the sophistication of these analyses might leave the impression that practicing levee engineers of the late 1980s and early 1990s were incapable of designing floodwalls able to survive the conditions that occurred during Katrina. Nothing could be farther from the truth. The principles of foundation design, then as now, rest primarily on the interpretation of subsurface information derived from suitably detailed boring programs, and from long experience with the response of soils and structures to the forces developed under critical conditions. These principles were well understood at the time these structures were designed.

We focus in the sections below on the pre-construction evolution of the floodwalls designed for the London Avenue Canal, first, because this canal experienced two breaches during Katrina, and, second, because the NOD has released far more documentation for the London Avenue canal than for the 17th Street Canal, which also breached.

Selection of I-walls or Inverted T-walls

Today, the I-wall segments of the drainage canal walls that breached are being replaced by inverted T-walls with sheet pile cutoffs driven to far lower tip penetration elevations than were in place when Katrina struck. Original GDMs for all of the drainage canals as well as for the Duncan Canal and IHNC called for a mixture of I-walls and the more robust T-wall structures. The extent of T-wall reaches proposed ranged from very limited segments around the pumping stations on the 17th Street Canal to virtually the entire east side wall on the London Avenue Canal, and all but about 290 ft of the Duncan Canal wall. T-walls were also specified for much of the west bank wall on the Orleans Avenue Canal, as well as portions of the east bank.

The criteria that govern the choice of an earthen levee or a floodwall, and the selection of an I-wall or T-wall were generally seen as reducing to one of space availability within the available or obtainable right-of-way. Mr. Al Naomi, a senior project engineer at the New Orleans District (NOD) responded to a question from a reporter at a post-storm press conference on September 2, 2005, about these choices by saying:

> "You go with the T-wall when you need a much higher wall than an I-wall will support. You go with an I-wall when you can't squeeze a levee in. In this particular case (17th St. Canal), the area around this canal is extremely densely populated and there wasn't any room to put a levee in. Certainly we prefer levees to I-walls or T-walls because they're somewhat less expensive and they're easier to construct. So the decision was, in this case, an I-wall…A T-wall would have required more technical work, more pile driving beneath the wall that would have been far more expensive, but not necessarily giving you any additional level of protection" (www.taxpayer.net/budget/katrinaspending/rndtble0902.pdf).

This statement gives a general idea of the thinking in the NOD. The Orleans Metro drainage canals include thousands of linear feet of earthen levees, T-walls and I-walls. Corps engineers have stated that earthen levees with the necessary crown elevations required too wide a base to fit into the right-of-way except near the mouths of the London Avenue and Orleans Avenue Canals (Figure 90). I-walls, whether constructed entirely of sheet pile or of sheet pile capped with concrete monoliths, are cantilever structures that require a levee for support (Figure 91), but the levee required is lower and therefore narrower at the base than an earthen levee with the same crown elevation. Inverted T-walls are pile-supported structures that may be embedded in an earthen levee, but do not require it for support (Figure 92). Both I-walls and T-walls are built atop sheet piles driven to a specified tip penetration elevation. The sheet piles act to cut off seepage under both wall types, but serve the additional purpose of providing cantilevered stability to the I-wall that is not required for the T-wall.

The primary message of Mr. Naomi's statement holds true. The NOD would build an earthen levee where it had space and an I-wall where there was too little right-of-way to construct a full-scale earth levee. If there was not enough space to build the lesser levee required for an I-wall, then a T-wall would be specified. There is, however, one additional aspect to the selection of wall type. If the sheet piles supporting the I-wall were required to be driven more deeply to provide protection against seepage or "piping" in a porous subsurface sand deposit, for example, then this might work to reduce stability. Longer sheet piles might flex or deflect too much under the load of water on the face of the exposed wall, creating the potential for a gap to form between

the levee and the wall. The IPET and ILIT have identified the formation of such a gap as contributing to the breaching on the 17th Street Canal floodwall, and possibly other I-wall failures, as will be discussed. The flexing or gapping problem does not arise with the T-wall sheet pile cutoff because it does not bear the load of the water against the exposed wall, which, instead, is resisted by a separate system of support pilings (Figure 92).

The last sentence of Mr. Naomi's post-storm response comparing cost and level of protection deserves more attention. T-walls are two to three times more expensive to build, particularly in the cramped quarters of the Orleans Metro drainage canals. Earthen levees already existed along these canals that could be incorporated into I-walls but offered less advantage for T-wall construction. The need to keep the canals flowing for drainage purposes, and to provide continued protection from hurricane surge during construction meant that T-walls had to be built inside sheetpile cofferdam structures that extended out into the canal. The most potential for bank instability occurs during the construction process when batter piles must be driven, or jetted using water pumped along the piles, at carefully determined angles. So, T-walls had to be carefully justified to win the cost-effectiveness competition with I-walls. They often lost.

It is instructive to note, however, all repairs of breached I-walls are now being made with T-walls.

Figure 90. Looking southeast from lakeshore at earthen levees in mouth of London Avenue Canal near the University of New Orleans campus.

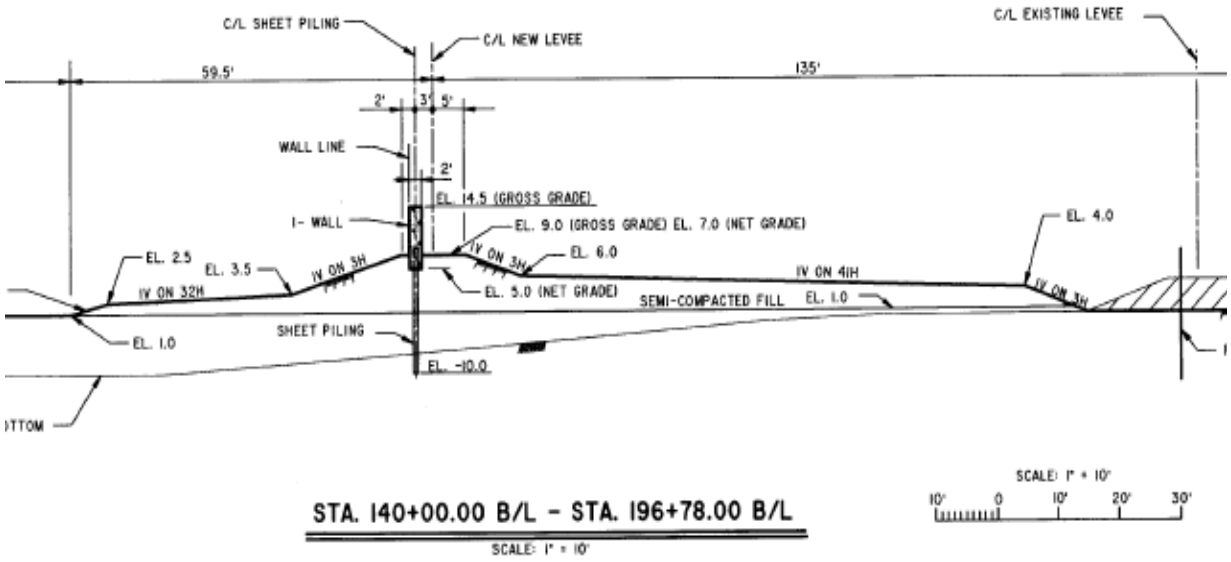


Figure 91. Typical I-wall design for St. Charles/Jefferson Return Levee from GDM 17A. Because a more extensive ROW was available, a new levee was constructed to support the cantilever structure that was set back from the pre-existing levee to the right.

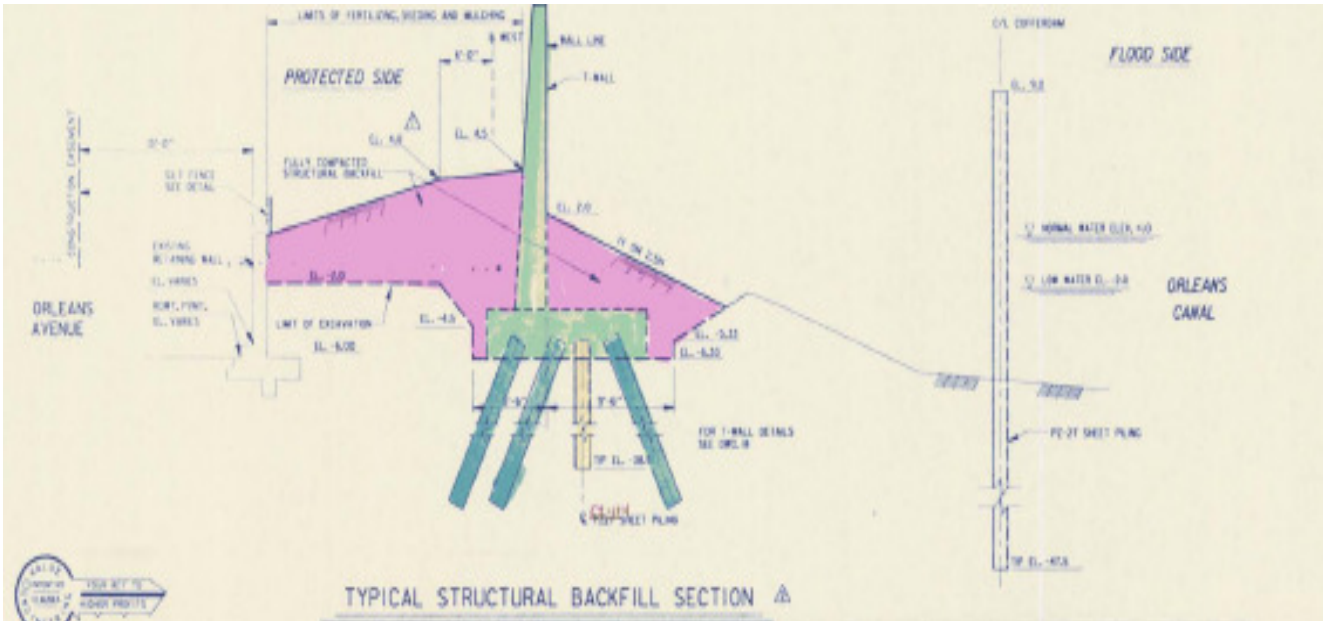


Figure 92. Typical inverted T-wall design for the Orleans Avenue Canal from as-built drawings. A sheet pile cofferdam used during construction is shown to the right. The pre-existing levee was degraded for installation of the T-wall and later reshaped. Note dark blue batter piles driven at an angle that support the base of the structure. Sheetpile wall extends straight down in orange. T-walls can be asymmetric or symmetric running the gamut from an inverted 'L' to 'T'.

I-walls failed in many locations during Katrina, and were sufficiently damaged in others that they are all now being scheduled for replacement. No T-wall structure failed, even though several were in difficult positions. A T-wall that is part of the wingwall closure on the west side of the Bayou Dupre control structure in the MRGO levee survived while adjacent concrete sheet piles were washed away (Figure 93).

As yet, over thirteen months after Katrina, the USACE has not made all as-built drawings for the floodwalls available, but it appears that the T-walls were built essentially as planned on the Orleans Avenue and Duncan Canals, yet were eliminated entirely on the London Avenue Canal. It is difficult, based on the documents available, to precisely reconstruct the rationale for eliminating the London Avenue Canal T-walls, which might have delayed or prevented the Mirabeau (south) breach. They were never proposed on the west bank in the vicinity of the north (Robert E. Lee) breach (Figure 94). The 1989 London Avenue Canal GDM 19A included the T-walls (Figure 95), as did a 1986 draft, but these were apparently superseded by a 1990 Revised GDM prepared for the Orleans Levee Board (Burk and Associates Inc. 1990). The T-wall proposed for the east bank in the 1989 GDM was replaced with an I-wall in the 1990 NOD Revised GDM (Figure 96). T-walls are entirely absent from the as-built drawings (Figures 97 and 98). The 1990 GDM includes the following statement indicating that an "interim" plan with a lesser level of protection than that specified in GDM 19A is to be pursued.

> "In April 1986, a general design memorandum was prepared for this flood protection project (protection for a 300 year storm plus two feet of free board) with the Corps' current geotechnical design standards which indicated a cost of 44 million dollars. At the time, the available budget for London Avenue Canal was considerably less than this amount. Therefore the Board decided to upgrade the existing system for interim flood protection to a level within the budgetary limits. Also, with the Corps standards being too stringent, the Board decided to follow the ASCE's state of the art geotechnical criteria and standards for structural design.
>
> As a result, this flood protection system was designed for a 100 year storm water elevation plus two feet of freeboard… Therefore, most of the proposed flood protection improvements will consist of providing "cantilever steel sheet pile I-wall floodwalls constructed in the existing earthen levees adjacent to the existing sheet piling" (Burk and Associates, Inc., 1990. London Avenue Canal Interim Floodwalls and Levees: Revised General Design Memorandum, Prepared for Board of Levee Commissioners, New Orleans Levee Board, p. I-1)."

Figure 93. Pile-supported inverted T structure that forms west side wingwall of the Bayou Dupre floodgate in the MRGO levee. Adjacent concrete sheetpiles were washed out but T-wall performed well.

The 1990 revised GDM calls for an additional foot of tip penetration, from -26 to -27 feet (NGVD29), in the east bank reach that includes the Mirabeau (south) London Canal breach (Figure 96). This may have been done to maintain the USACE required sheet pile penetration to head ratio at the recommended 3:1 value, where the design surge elevation on the exposed wall provides the head. **It does contain a curious rationale for lowering the crown level by 1 foot from the 1 in 300 year hurricane, which is interpreted as the 1959 SPH still water level, to a supposed 1 in 100 year level.** We have discussed the spurious derivation of this return frequency in an earlier chapter. The analysis contained in the 1990 Revised GDM (Plate II-2) shows how seriously such claims by the Corps of Engineers were taken, however, and how they were used to argue for lowering an already deficient level of protection still farther below that authorized. The reference to ASCE geotechnical criteria that are presumably less conservative than those of the USACE is also interesting, if vague.

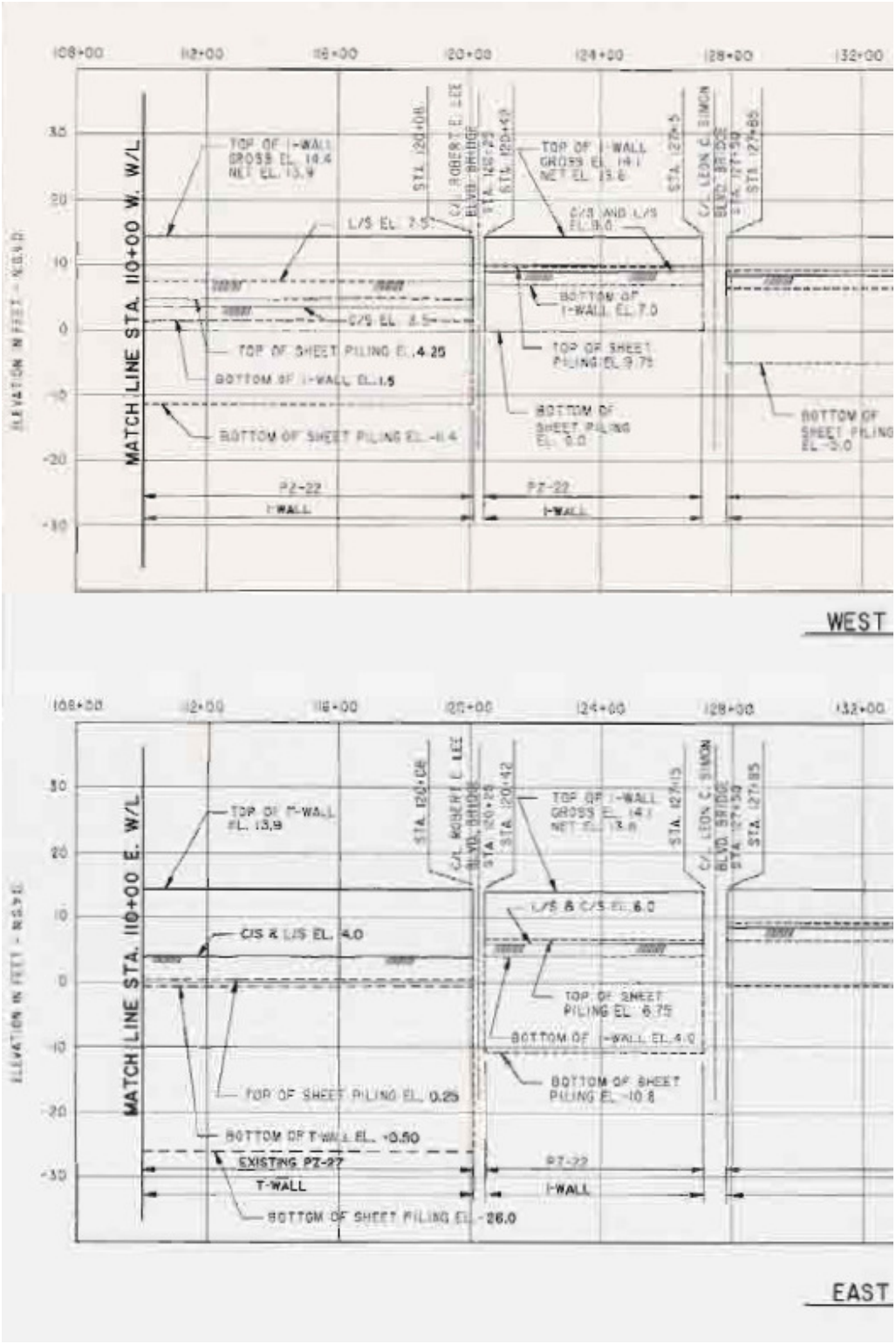


Figure 94. Portion of design drawing from 1989 USACE GDM 19A showing a T-wall with proposed sheet pile tip penetration to -26 ft (NGVD29) to be constructed on the east bank (bottom panel) of the London Avenue Canal opposite of the north breach site on the west bank at station 118, just south of the Robert E. Lee Boulevard bridge (top panel). An I-wall is proposed for the breach location with proposed sheet pile tip penetration to -11. 4 ft.

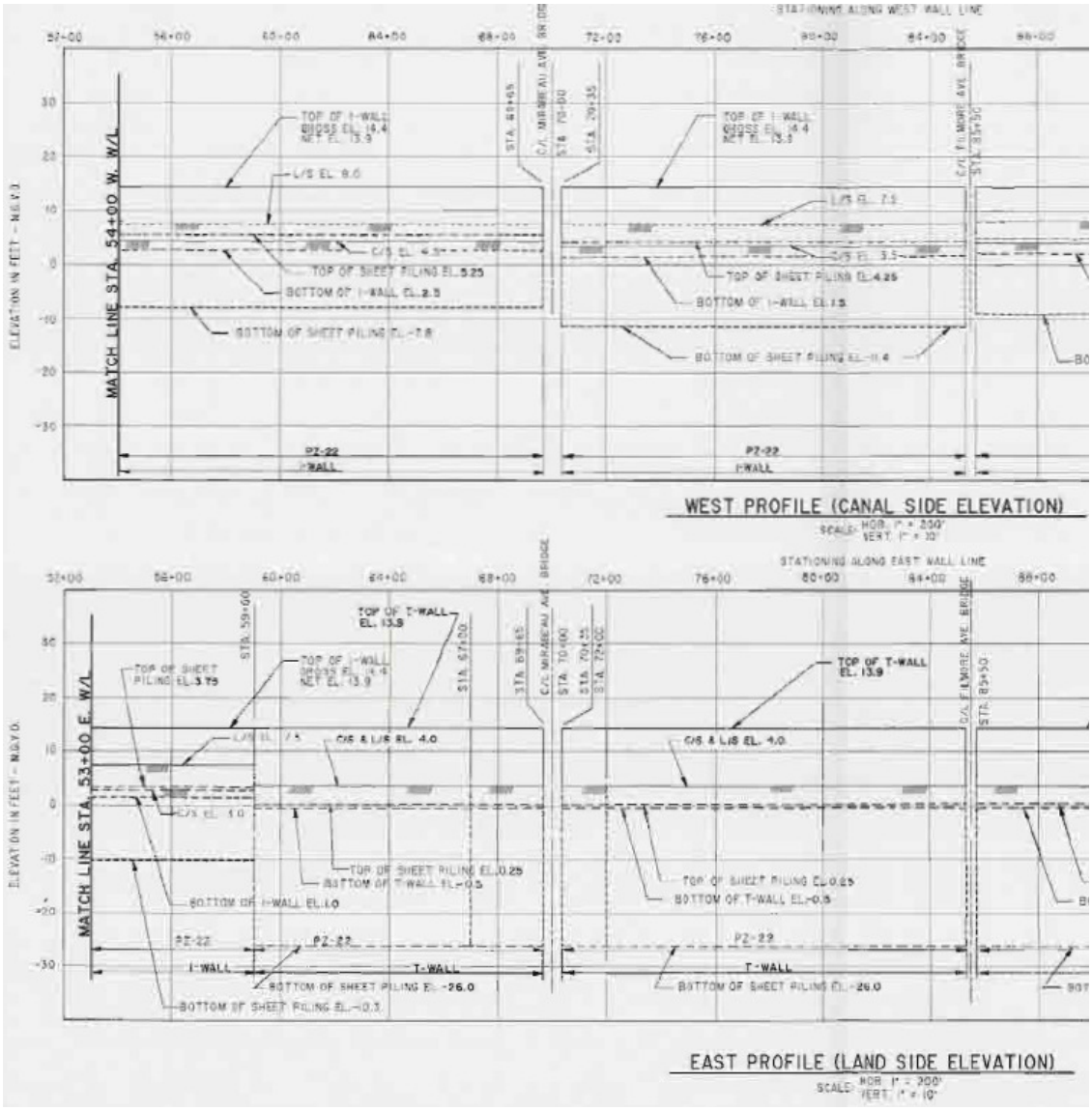


Figure 95. Portion of design drawing from 1989 USACE GDM 19A showing a T-wall to be built on the east bank of the London Avenue Canal (lower panel) between the Mirabeau bridge and station 75 at the location of the Mirabeau (south) breach with a proposed sheet pile tip elevation of -26 ft (NGVD29). An I-wall is proposed for the opposite (west) bank (upper panel) with sheet pile tip penetration to -11.4 ft (NGVD29).

Selection of Sheet Pile Type and Tip Penetration Elevation for I-Walls

Documents recording stages of the pre-construction evolution of the London Avenue Canal floodwalls give insight into another important aspect of cantilever I-wall design. The levee into which the sheet pile is driven provides the necessary stability for any sheet pile wall, but the concrete cap is not always used. In fact, uncapped sheet pile walls provided interim hurricane protection along all of the Orleans Metro drainage

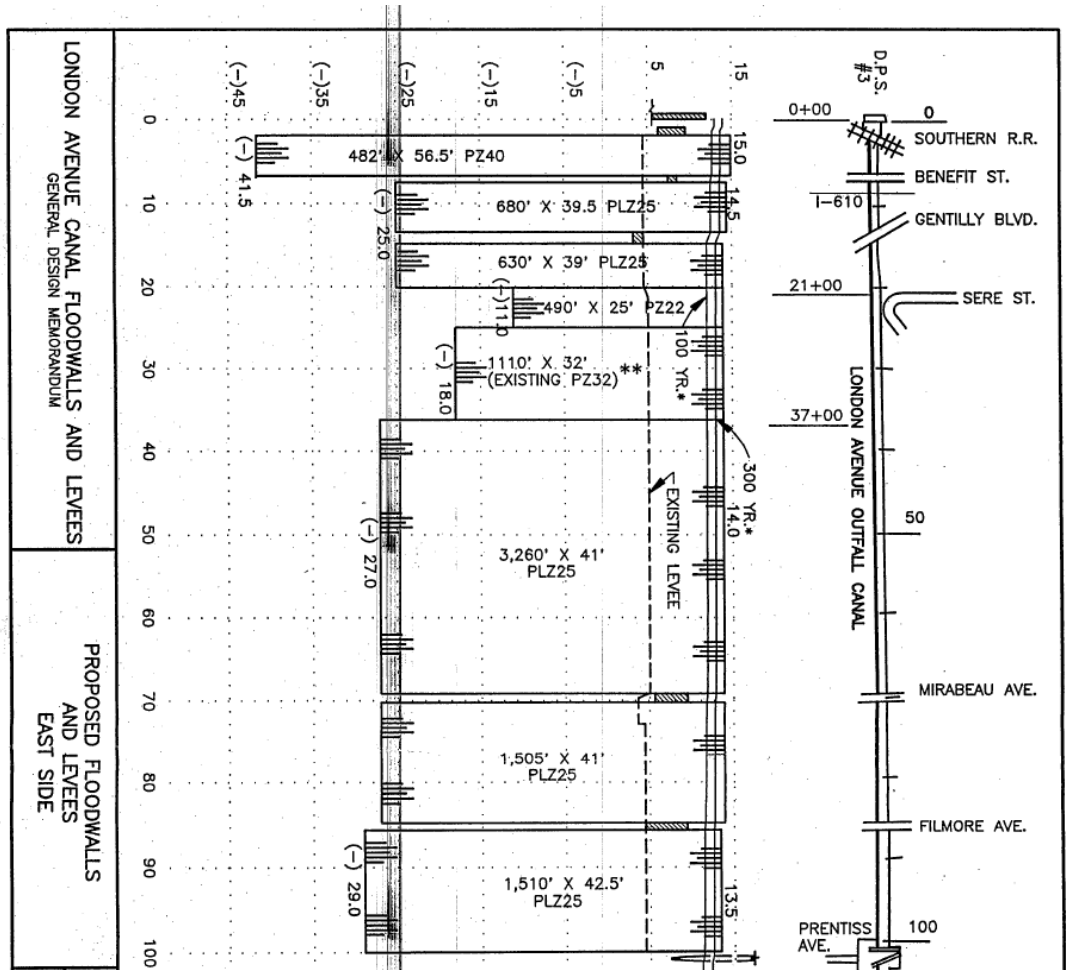


Figure 96. Portion of design drawing from 1990 Revised GDM (Burk and Associates, Inc. 1990) showing east bank I-walls at location of Mirabeau (south) London Avenue Canal breach (station 75) and 100 and 300 year crown levels of protection, but showing sheet pile tip penetration in breach location to -27 ft (NGVD29).

canals for a number of years before projects were approved to raise the level of protection for the GNO HPS, following adoption of the 'high-level' plan. Many sheet pile walls in more remote areas of the GNO HPS are not capped with concrete panels or monoliths (Figure 99), but the concrete capping provided a more aesthetically pleasing approach favored for use in urban areas.

I-wall construction involved some preparation of the existing levees, and a decision about whether to use existing sheet pile or drive new sheets. Finally, forms were built and the reinforced concrete panels were cast in place over the top of the exposed sheet pile with steel reinforcing to ensure a rigid connection between the sheet pile and the monolith (Figure 100). While the sheet piles are made to interlock along the sides, the reinforced concrete panels that extend the wall to the desired crown height cannot be continuous laterally to allow for thermal expansion and contraction as designed. A

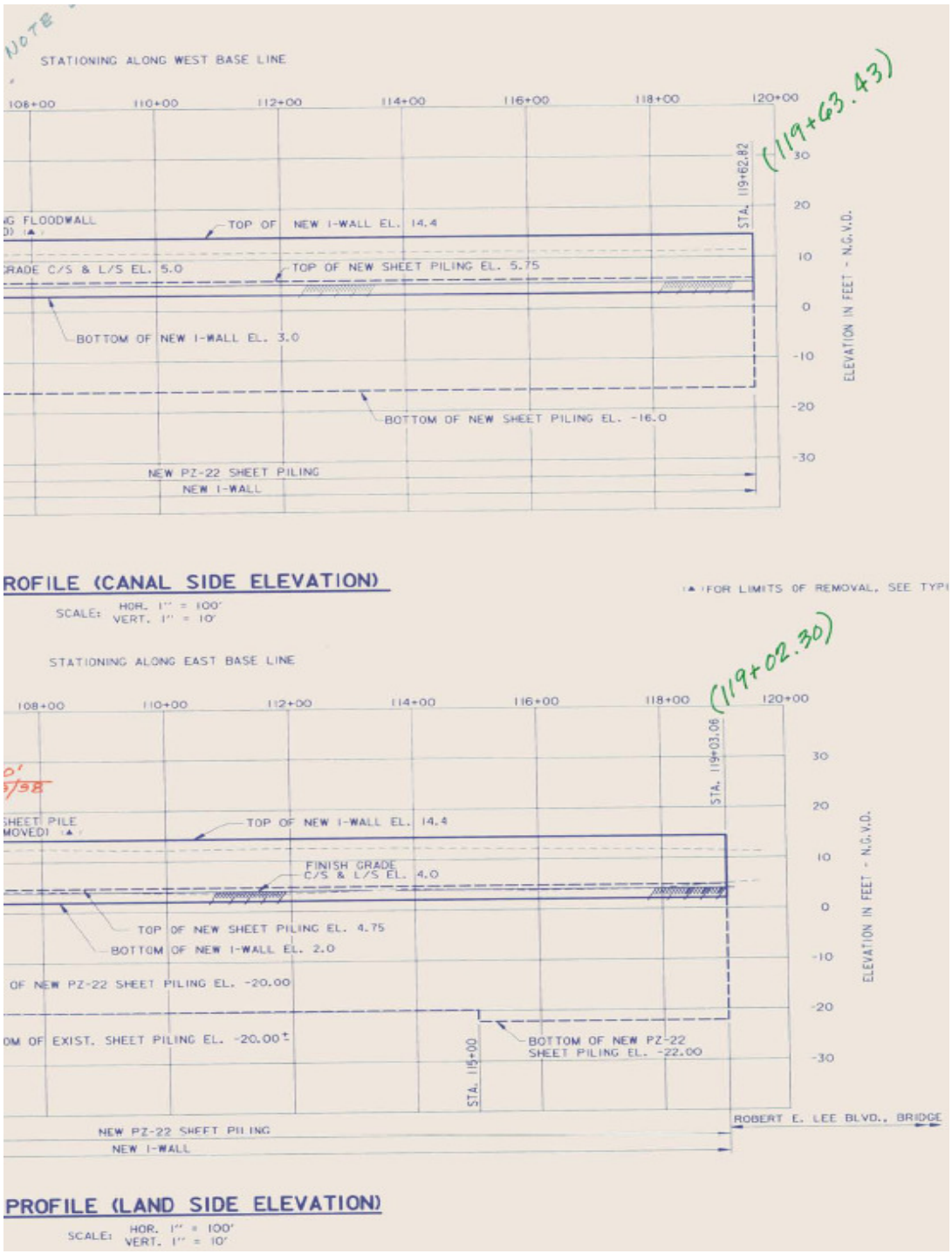


Figure 97. As-built drawing for east (lower panel) and west (upper panel) London Avenue Canal floodwalls in the vicinity of the north Robert E. Lee breach at station 118 on the west side showing that I-wall has been substituted for T-wall on east bank, and the sheet pile tip penetration elevation on the west side in the breach location has been extended from -11.4 (GDM 19A) to -16 ft (NGVD29).

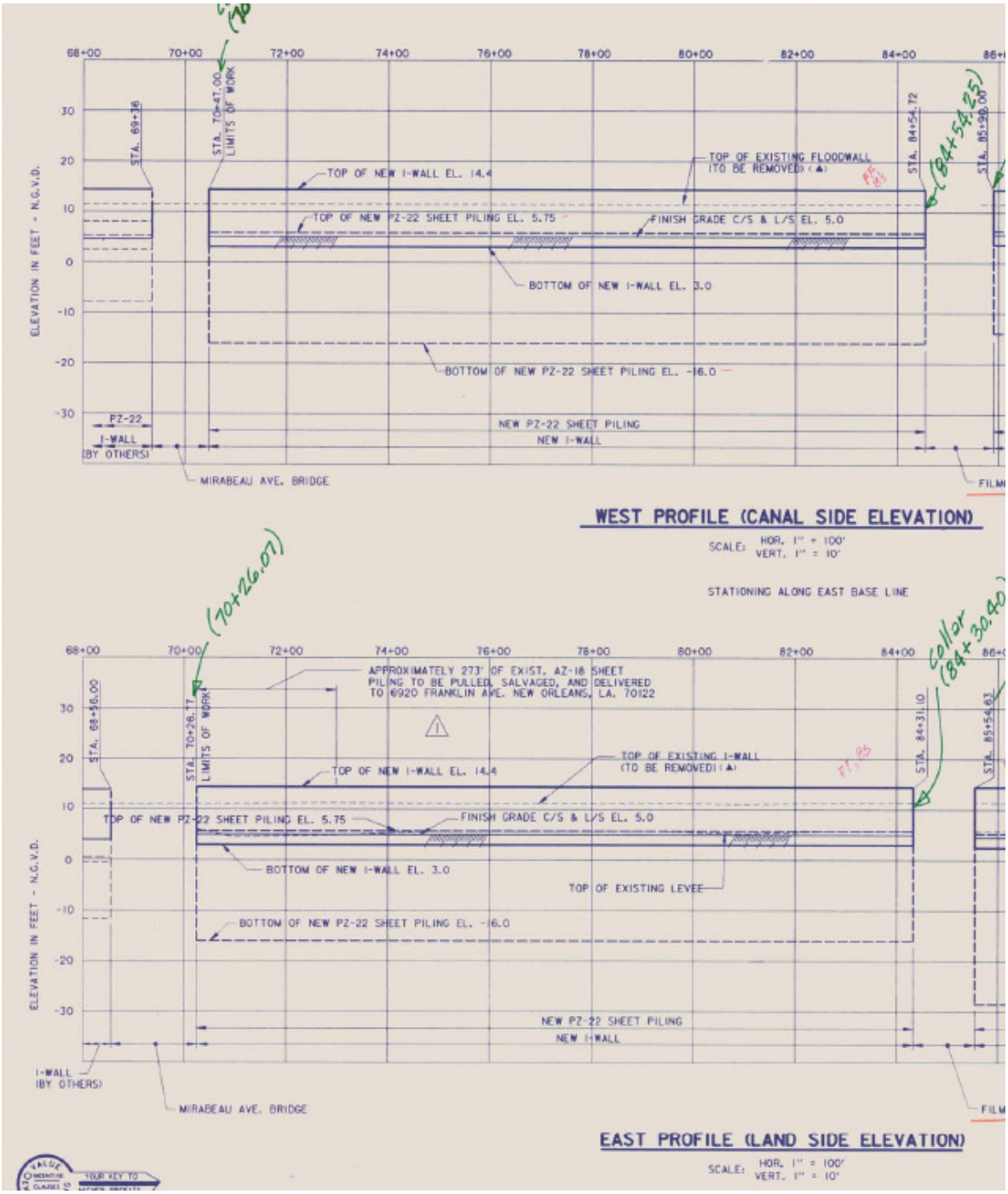


Figure 98. As-built drawing for east (lower panel) and west (upper panel) London Avenue Canal floodwalls in the vicinity of the south (Mirabeau) breach at station 75 on the east side showing that I-wall has been substituted for T-wall on east bank, and the sheet pile tip penetration elevation has been raised (shortened) at the breach location from -26 (GDM 19A) to -16 ft (NGVD29).

rubber waterstop is used to provide the necessary flexible seal between panels (Figure 100). When the concrete caps are added, skin friction on the piles must support the weight of the monoliths without settling, and provide enough rigidity to prevent them from deflecting and moving independently.

Sheet piles used in the GNO HPS were typically characterized by the type of steel, whether cold- or hot-rolled, the thickness or gauge of the steel, the shape of folds when viewed end-on (i.e. 'S' or 'Z'), and the type of interlock. All of these characteristics affect the rigidity of the interlocking wall formed, the depth to which it can be driven and, of course, how much it costs.

Sheet pile tip penetration and the penetration to head ratio (P to H ratio) were touched on previously. It appears from the 1994 as-built London Avenue Canal drawings that the Corps followed through with the design changes in the 1990 Revised GDM; I-walls were substituted for T-walls on the east bank (Figure 98). Another significant design change from both the 1989 GDM and the 1990 Revised GDM is apparent in the as-built drawings. The sheet pile tip penetration elevation in the east bank Mirabeau breach area has been reduced from -26 or -27 ft to -16 ft (NGVD29).

Figure 99. Damaged sheet pile wall used to raise the level of flood protection on the MRGO levee. Note the interlocks between the sheets and the folds added to increase rigidity.

Figure 100. Damaged connection between top of sheet pile and two I-wall monoliths at the north London Avenue breach. Reinforcing steel bars that are cast into the concrete are welded to the top of the sheet pile. Twisted rubber gasket that forms the waterstop between the monoliths is visible above the top of the sheet pile.

A likely rationale for the reduction in sheet pile penetration at the Mirabeau breach site appears in a 1993 report by Eustis Engineering for Burk and Associates, Inc., discussing additional geotechnical analyses (Eustis Engineering 1993). A table displays results of various standard criteria applied to set the depth, or, alternatively, tip elevation, for I-wall sheet pile penetration required to assure that the wall will remain vertical when subjected to the design load or specified canal water level (Figure 101). These same criteria are repeated in every GNO HPS GDM that includes I-walls. Shear strengths are assessed using a Q-case test and an S-case test with an appropriate factor of safety applied. Q stands for shear under an instantaneous or 'Quick' loading that is established based on unconsolidated-undrained triaxial shear tests of boring samples analyzed in the laboratory. S stands for shear under a 'Slow' loading that is derived from consolidated-drained triaxial shear tests on clays or from standard penetration tests on sands in the field.

Minimum sheet pile penetrations necessary to assure that the cantilever wall will not lean or deflect were then calculated for each of these conditions using the USACE CWALSHT computer program (Figure 101). USACE practice generally allows the designer to specify the lesser of the Q- or S-case penetration depths as long as it does

not produce a penetration to head ratio less than 3:1, as was discussed above. The table from the 1993 Eustis Engineering report gives values in the first row derived from the two test cases and from the minimum penetration to head (P to H) ratio for the east bank reach containing the Mirabeau Avenue breach (Figure 101). The tip elevations specified for this reach for the Q-case, S-case and P to H Ratio are -4.0, -28 and -16 feet, respectively. The required factors of safety are shown in the notes at the bottom of the table. The soils under the Mirabeau breach are shown in another figure from the same report to be beach sand from elevation -10 to -45 ft. The grade of the surrounding neighborhood is approximately -4 ft. The P to H ratio yields the -16 ft (NGVD29) tip penetration elevation that appears in the as-built drawings for the Mirabeau breach site and was apparently selected for construction following standard USACE practice (Figure 98). Eustis (1993) states that "in every case, the ratio of penetration to head governs the design sheet pile tip elevation."

The approach described for selecting the sheet pile tip elevation appears to be almost menu-driven even if it does follow general USACE policy for sheet pile stability. Practicing geotechnical engineers consulting with Team Louisiana were particularly uncomfortable with this "one-size-fits all approach" to the London Avenue Canal floodwall design. They believe both the Q-case test and the P to H ratio produce less than conservative results, because the porosity of the thick Pine Island sand stratum ensured that the hydrostatic head developed in the canal would be felt almost immediately after construction, against the sheet pile. This would occur because the bed of the London Avenue Canal adjacent to the I-walls penetrates the top of the beach sand stratum (Figure 102). Given the porosity, the Corps should have considered the shear under the S-case which would have meant sheet pile -28 to -39 feet. Geotechnical engineers have learned that it is wiser to consider the most likely failure mode in a particular situation when designing a flood protection structure than to rely on general policy, particularly when the structure protects a high-value urban area. Sheet pile stability is only one factor that is considered in I-wall design. HQUSACE guidance on construction of sheet pile walls (EM 1110-2-2504), most recently issued in 1994, lays out the standard of practice for seepage analysis at the time the London Avenue Canal walls were being designed.

> "Where seepage occurs, the differential water pressure is dissipated by vertical flow beneath the sheet pile wall. This distribution of the unbalanced water pressure can be obtained from a seepage analysis. The analysis should consider the permeability of the

LONDON AVENUE OUTFALL CANAL
PROPOSED I-WALLS AND T-WALLS
MIRABEAU AVENUE TO LEON C. SIMON BOULEVARD
NEW ORLEANS, LOUISIANA

SUMMARY OF I-WALL ANALYSES

BANK	STATIONS	FINISHED GRADE*		OFFSET IN FEET	SOIL REACH	TIP ELEVATION IN FEET, NGVD			
		P.S.	F.S.			○	□	◇	△
East	70+24.93 to 84+36.60	5.0	3.0	+4	1	-4.0	-28.0	-16.0	-6.5
	85+54.63 to 95+00	4.5	2.5	Existing Sheetpile	2	-6.2	-36.0	-18.0	-8.0
	95+00 to 99+18.63				1		-32.0		
	102+62.50 to 115+00	4.0	2.0	-4	1	-9.5	-35.0	-20.0	-9.0
	115+00 to 119+16.17				2		-39.0		
	120+40.00 to 126+65.00	5.5	5.5	+4	2	-2.7	-36.0	-14.0	-4.5
West	70+64.00 to 84+90.00	5.0	3.0	+1	1	-4.0	-28.0	-16.0	-6.5
	86+10.00 to 95+00	5.5	3.5	+1	2	-2.6	-36.0	-14.0	-5.5
	95+00 to 101+12.19				1		-26.0		
	101+12.19 to 115+00	5.0	3.0	+1	1	-4.0	-28.0	-16.0	-6.5
	115+00 to 119+65.00				2		-38.0		

+ Levee Crown. (PS) = Protected Side; (FS) = Flood Side.
* From existing floodwall. (+) = toward protected side; (-) = toward flood side.
○ Q-Case: Factor of Safety = 1.5 with SWL or Factor of Safety = 1.0 with SWL + 2
□ S-Case: Factor of Safety = 1.5 with SWL
◇ Ratio of Penetration to Head = 3 (Controls design elevation in all cases)
△ Creep Length using Lane's Weighted Creep Ratio

FIGURE 2

EUSTIS ENGINEERING COMPANY, INC.

Figure 101. Table from report detailing additional geotechnical analyses for the London Avenue Canal I-walls (Eustis Engineering 1993). The Mirabeau breach is at station 75 on the east bank that is included in the reach described in the top row. The north breach is at station 118 on the west bank in the reach described in the bottom row.

> surrounding soils as well as the effectiveness of any drains if present. Techniques of seepage analysis appropriate to sheet pile wall design include flow nets, line of creep method and method of fragments. These simplified techniques may or may not yield conservative results. Therefore, it is the designer's responsibility to decide whether the final design should be based on a more rigorous analysis, such as the finite element method. Upward seepage in front of the sheet pile wall tends to reduce the effective weight of the soil, thus reducing its ability to offer lateral support. In pervious material the effects of upward seepage can cause piping of material away from the wall, or, in extreme cases, cause the soil to liquefy. Lengthening the sheet pile, thus increasing the seepage path, is one effective means of accommodating seepage" EM 1110-2-2504, p. 4-6).

From the above it is clear that in porous soils such as the sands beneath London Avenue Canal, the sheet pile lengths should have been long enough to stop seepage, period! The 1993 Eustis Engineering report includes the results of a creep distance analysis using Lane's Weighted Creep Ratio method in the last column of the summary table (Figure 101). The value given is -6.5 feet, which gives the impression that seepage should not be a serious factor. In a 1986 report, Eustis had previously calculated a Lane's Weighted Creep Ratio for stations 37 to 120 (east and west banks) of 3.2, which is noted as indicating a creep distance of 57 feet (Figure 103).

Seepage and Lane's Weighted Creep Ratio Calculations

Van Zyl and Harr (1981) classified seepage erosion failures into three modes: heave, piping and internal erosion. In order to investigate the hydraulic features of a weir or dam (levee), especially seepage, extensive field studies were made by Bligh (1910) on weirs in Egypt and India. However, Bligh's method was too conservative and was later modified by Lane (1935) on the basis of a study of about 280 existing dams on pervious foundations, 24 of which had failed. The "line of creep" theory is based on the conclusion that the "line of creep," i.e., the line of contact between the dam (levee) and cutoffs (sheet pile) with the foundation will produce less resistance to percolation than another path through the foundation material or substrate (National Engineering Handbook 1968). Thus Lane believed that the majority of dam failures he had investigated were due to piping along the line of creep.

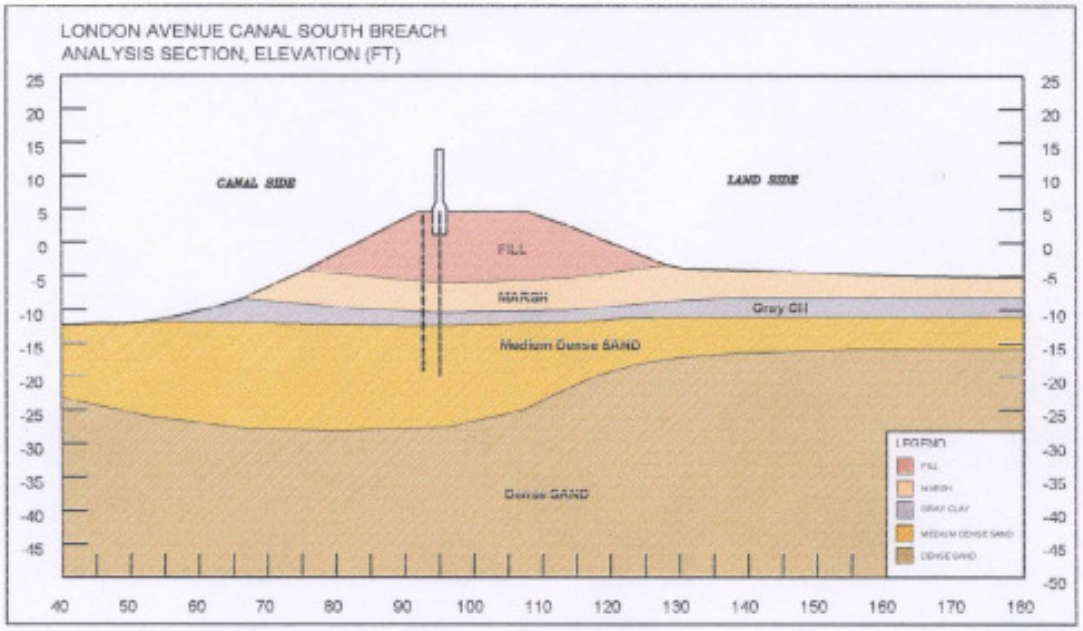


Figure 102. Geological cross-section prepared by ILIT (2006) through Mirabeau (south) breach on London Avenue Canal looking north. Note that canal bed bottoms in the Pine Island sand, and that the sheetpile of the I-wall extends into the sand but does not penetrate through it.

52. Underseepage. Underseepage of the recommended sheetpile wall sections was generally evaluated based on Lane's Weighted Creep Ratio Method of Analysis. The results of these computations are tabulated below.

Reach	Weighted Creep Distance In Feet	Net Head In Feet	Lane's Weighted Creep Ratio
I	75	11.9	6.3
I*	46	11.9	3.8
II	56	14.9	3.8
III	57	17.9	3.2
IV (Eastside)	50	16.6	3.4
IV (Westside)	26	11.6	2.3
V**	26	11.5	2.3
V***	57	11.5	4.9

* Alternative Design

** Sta. 127+00 to 144+50 (Eastside);
Sta. 127+00 to 142+50 (Westside)

*** Sta. 147+50 to Lake (Eastside);
Sta. 145+00 to Lake (Westside)

Figure 103. Excerpt from 1986 Eustis Engineering geotechnical report for London Canal floodwalls and levees discussing underseepage potential. Reach III in this report covers the entire area of interest on east and west banks from Station 37 to 120.

Lane's method incorporates an assumption that the resistance to flow through the soil, and therefore the rate of pressure drop, will be three times higher in the vertical than in the horizontal direction. One explanation put forward is that horizontal layering in the soil provides an easier passage to seepage. Thus in Lane's method, vertical structural surfaces are weighted so that they become three times as effective in reducing seepage pressure as horizontal ones. Many researchers have pointed out that this assumption holds true for limited set of soil conditions. It certainly does not hold true in very sandy substrate, especially well rounded and sorted beach sands, such as typifies the Pine Island sands beneath the London Avenue Canal in New Orleans.

For this reason Lane's should be used with caution, and this and other empirical methods are commonly used only in the routine design of low- to medium-head structures outside of urban settings. Lane's Theory is simple and rapid to apply so it is convenient for making quick preliminary analyses of a structure's overall resistance to seepage. However, the method does not provide any check against the danger of excessive pressure gradients developing locally at the downstream end of the structure. It also appears to imply that a vertical cut-off would be equally effective whether located at the upstream or downstream end of a structure. More detailed analytical methods show this is not the case. It should be remembered that the line-of-creep approach is empirical and like every empirical approach leads to a design with an unknown factor of safety.

Lane's creep ratio is determined by a so-called safe seepage length 'L' divided by h the head of water:

$$CR_{Lane} = L/h$$

L is defined as the sum of lengths of vertical flow paths plus one third of the sum of horizontal flow paths with surface sloping less than 45 degrees being considered horizontal; sloping greater than 45 degrees being vertical, so:

$$L = \Sigma \text{ lengths vertical flow paths plus } 1/3(\Sigma \text{ of horizontal flow paths})$$

Lane (1935) then gives a table of recommended or safe creep-head ratios L/h. So, for very fine sand or silt if the ratio is greater than 8.5 the design is considered safe; for fine sand the ratio needs to be greater than 7.0; for medium sand greater than 6.0; and for soft clay 3.0 (Adapted from USBR 1974, Untitled Table, Page 364).

Eustis Engineering (1986 and 1993) used the Lane's Creep Ratio in some of their calculations for London Ave canal. First they determined the Lane's creep ratio for the various reaches and then used the creep ratio as a check on the sheet pile lengths (Figures 101 and 103). For the I-wall reach at the Mirabeau breach, Eustis state the head is 17.9 feet, which appears to be the 13.9 height of the wall, added to the fact that the land behind the levee on the protected side is at minus 4 feet, giving h = 17.9 feet. They used a weighted creep length of 57 feet and determined a Lane's creep ratio of 3.2.

So how long must the sheet pile have been to come up with a creep ratio of 3.2? The width of the levee at the fill marsh surface is 45 feet (Figure 102). To obtain a ratio of 3.2 there would have to have been a vertical flow path of 39 feet (57-55/3 = 57-18 = 39). Thus they must have done this calculation assuming a sheet pile length of 18.5 feet below the base, which is at minus 4; thus the sheet pile must have extended down to -22.5 feet NGVD29. If there was no sheet pile then the levee would have had to be 5 x 73 = 171 feet wide

Assuming the material to be soft clay then the creep ratio they calculated exceeds Lane's recommendation by 6%, which would be cutting it very, very close. However they fail to recognize that they are only taking into account the thin marsh/clay layer above the very thick sands (Figure 102). As any geologist or geotechnical engineer would know, the sands would be the preferred pathway for any seepage. If they had done the same calculations for the sands with a ratio of 8.0 then the weighted creep distance flow path would have been 125 feet too short, thus sheet pile to -66.5 feet NGVD29 would have been required. This is a huge and very significant difference. **The sand calculation would have required that the sheet pile be 44 feet deeper to form an effective seepage barrier.**

In a 1993 report (Figure 101), Eustis use Lane's Weighted Creep ratio to determine what the sheet pile length should be. For the same Mirabeau reach as calculated above they state they need sheet pile to -6.5 feet. So something has changed between 1986 and 1993! Using sheet pile to -6.5 and the same levee cross section as above (Figure 102) gives a Lane's weighted creep ratio of 1.2, which is well below that recommended by Lane for soft clays, let alone for sands.

As pointed out above, Lane's method has been widely used although there are obvious problems with some of its assumptions. First, flow configuration and gradients are not affected by the soil type (Kashef 1986) in homogeneous and isotropic media under steady–state flow. The gradients are affected by the

quantity h, and so are the uplift pressures. The National Engineering Handbook (1968) categorically states that so many intermediate variables affect the design for safety against seepage or piping that a large factor of safety is mandatory. When one considers that the hydraulic conductivity can vary from 10 cm/sec for coarse gravel to 10^{-9} cm/sec for dense clay, the complexity of the problem is apparent. If different materials are present in the subsurface or foundation, the material having the largest creep ratio should be considered as the foundation material and the design made accordingly. It seems almost unnecessary to point out (Nation Engineering Handbook 1968) that adequate foundation investigations are mandatory for safe design of structures that protect cities.

Uplift or Heave Forces

Soil type, rate of head increase and the flow condition are the main dependents for modes of seepage erosion failure (Van Zyl and Harr 1981). The soil type controls whether heave is followed by a quick condition as in clean sand or whether heave leads to crack formation, concentrated flow and piping. Heave, leading to cracks, concentrated flow and piping, appears to be more common in granular soils with a large percentage of fines. A rapid increase in head may result in heave of the surface, leading to a quick condition (Van Zyl and Harr 1981). This would be a likely failure condition on the downstream side of a levee bounded canal that is being filled rapidly, such as occurred during Katrina.

Heave is analyzed by comparison of seepage force per unit volume with effective unit weight of selected critical volume of soil. Terzaghi (1929) presented an exit gradient approach to seepage analysis in his classical work on failure of dams by seepage erosion. His theoretical development was based on the summation of the vertical seepage forces exerted by the upward flow of water and the vertical downward weight of the submerged soil. However there is an even simpler method.

It is imperative in the design of levees and flood walls to determine the uplift forces on the downstream (protected side) to ensure that these do not exceed the load (weight) capacity of the floor and thus impact the overall stability. The ratio of the downward forces to the uplift forces should be greater than 1.0 by an appropriate margin of safety. Kashef (1986) amongst others points out that the factor of safety in hydraulic structures should not generally be less than 4. So just what was the ratio for the London Avenue canal flood wall at Mirabeau prior to the breach?

A very simple calculation allows one to determine the ratio between uplift vs. downward forces on the protected side of the levee. Recognizing that the thick well sorted beach sands beneath the floodwall would rapidly allow seepage (as shown by the Lane's creep calculations for example), the pressure at the base of the clay layer/top of the sand in the canal due to the surge can be calculated as:

> head x density of water = hsurge+depth to interface x 62.4 lbs/cub ft = **1280 psf** where the hsurge is 10.5 feet and the top of the sand is at -10 ft

On the protected side 6 feet of marsh and clay overlies the sand layer, so the:

> gravity (weight) downward force = 6 feet x density of the soil which is given as 110 lbs/cubft = **660 psf.**

So the upward pressure far exceeds the downward, the ratio being 0.5. As pointed out this should be at least 4 so it is easy to see that the system as built would fail, there being no cutoff (sheetpile) as a hydrologic/seepage barrier in the sands.

The Corps of Engineers has done extensive studies on seepage, piping and sand boils. Additionally, the levee designers could easily have found information on seepage and creep ratios in PCA (1980), and USACE EM 1110-2-1901 (1993). The applicability of Lane's Creep ratio for a specific structure should always be questioned and only used as a rule of thumb. Given that thousands of lives depended on the levee system, additional seepage and piping analyses using more rigorous methods (e.g. flow nets) should have been undertaken to assess hydraulic gradients, uplift pressures, and seepage flows. Measures to control seepage normally include provisions to increase the length of the seepage path and otherwise reduce seepage flows, and provisions to filter and drain the seepage water. Given the high importance of controlling seepage on the safety and stability of the levees, adequate redundancy should be incorporated when designing seepage control measures.

The above review using simple techniques available to the engineers who designed the London Avenue canal levees proves that the designs were inappropriate and the factor of safety based on uplift forces was 0.5! No wonder the system failed.

Development of Underseepage and Sand Boils

We believe that the failure at Mirabeau resulted from uplift forces cracking the overlying clay, the subsequent development of large sand boils which expanded rapidly into a large scour hole into which the levee flood wall collapsed. This failure mechanism has been described before for the Mississippi River system. Turnbull and Mansur (1961) explained underseepage mechanisms and sand boil formation at Mississippi River levees as a result of the studies and investigations conducted by the Army Corps of Engineers covering a period of 1937 to 1952:

> "Whenever a levee is subjected to a differential hydrostatic head of water as a result of river stages higher than the surrounding land, seepage enters the pervious substratum through the bed of the river and riverside borrow pits or the riverside top stratum or both, and creates an artesian head and hydraulic gradient in the sand stratum under the levee. This gradient causes a flow of seepage beneath the levee and the development of excess pressures landward thereof. If the hydrostatic pressure in the pervious substratum landward of the levee becomes greater than the submerged weight of the top stratum, the excess pressure will cause heaving of the top blanket, or will cause it to rupture at one or more weak spots with a resulting concentration of seepage flow in the form of sand boils."

> "In nature, seepage usually concentrates along the landside toe of the levee, at thin or weak spots in the top stratum, and adjacent to clay-filled swales or channels. Where seepage is concentrated to the extent that turbulent flow is created, the flow will cause erosion in the top stratum and development of a channel down into the underlying silts and fine sands, which frequently exist immediately beneath the top stratum. As the channel increases in size or length, or both, a progressively greater concentration of seepage flows into it with a consequent greater tendency for erosion to progress beneath the levee."

> "The amount of seepage and uplift hydrostatic pressure that may develop landward of a levee is related to the river stage, location of seepage entrance, thickness and perviousness of the substratum and of the landside top stratum, underground storage, and geological features. Other factors contributing to the activity of the sand boils caused by seepage and hydrostatic pressure are the

> degree of seepage concentration and the velocity of flow emerging from the boils."

It is not possible to determine how the analysis changed between 1986 and 1993. As is noted in HQUSACE guidance for seepage analysis (EM 1110-2-1901, p. 4-9), first issued in 1986, USACE-approved computer programs have been available since at least 1980 for finite-difference and finite-element analysis of seepage problems in embankments. There is no evidence that these more rigorous approaches were applied or required by the USACE for the London Avenue Canal floodwall design, even though it was clearly warranted in this case.

The reliability of any cantilever I-wall used for flood protection ultimately depends on the stability of the levee in which it is embedded, as has been discussed above. This was typically assessed for GNO HPS applications using one of the USACE accepted slope stability computer programs. At the time that the London Avenue Canal floodwalls were under design, the NOD generally required application of the LMVD Method of Planes. The 1993 Eustis Engineering report provides a summary of results of analyses using this technique for the east wall in the vicinity of the breach at Mirabeau and for the east wall opposite of the west bank breach that experienced distress short of complete failure (Figure 104). The driving and resistance forces are summed and compared above a potential slip-plane failure surface that yields the lowest factor of safety (sum of resisting or shear forces (R) divided by sum of the driving forces (D) causing movement = R/D (Figure 105)).

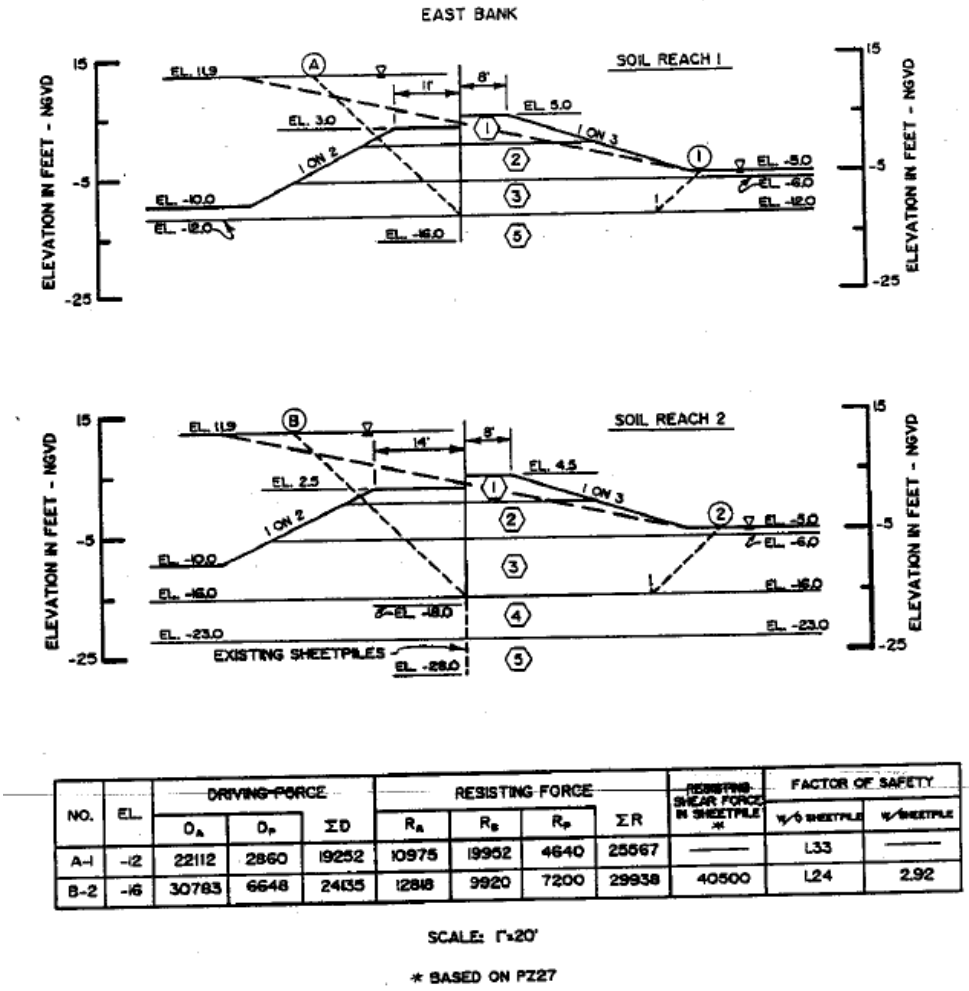
The levee supporting the proposed I-wall in the Mirabeau breach section is shown in the summary table (A-1) as stable (FS > 1.3) at a canal level of 11.9 ft (NGVD29), without adding resistance from the sheet pile (figure 105). Some resistance from sheet pile was required to attain an acceptable factor of safety for the northern reach of the proposed I-wall (B-1). **It is worrisome that it was considered acceptable to include sheet pile resistance to attain an acceptable factor of safety for levee stability, because it is the levee that supplies the fundamental support of the cantilever wall, rather than the reverse.** EM 1110-2-2504 (1994) discusses failure associated with a deep-seated rotation of the entire levee mass as follows.

> "This potential failure (deep-seated) is independent of the structural characteristics of the wall and/or anchor. The adequacy of the system (i.e. factor of safety) against this mode of failure should be assessed by the geotechnical engineer through

Figure 104. I-wall leaning toward landside on east bank of London Avenue Canal south of Robert E. Lee Boulevard Bridge across canal from north breach.

> conventional analyses for slope stability (EM 1110-2-1902). This type of failure cannot be remedied by increasing the depth of penetration or by repositioning the anchor. The only recourse when this type of failure is anticipated is to change the geometry…or improve the soil strengths" (EM 1110-2-2504, p. 5-1).

It can be generally stated that the Orleans Metro drainage canal levees are smaller and narrower than would be ideal for I-wall construction. The dimensions were accepted as a design constraint that could be addressed by adjusting the sheet pile penetration, rather than a signal that another type of structure might be required.



NO.	EL.	DRIVING FORCE			RESISTING FORCE				RESISTING SHEAR FORCE IN SHEETPILE *	FACTOR OF SAFETY	
		O_A	O_P	ΣO	R_A	R_B	R_P	ΣR		w/o SHEETPILE	w/ SHEETPILE
A-1	-12	22112	2860	19252	10975	19952	4640	25567	——	1.33	——
B-2	-16	30783	6648	24135	12848	9920	7200	29938	40500	1.24	2.92

SCALE: 1"=20'

* BASED ON PZ27

Figure 105 Eustis Engineering (1993) results of east bank London Avenue Canal slope stability analyses for the Mirabeau breach area (upper panel) and the distressed wall opposite the Robert E. Lee (north) breach (lower panel). The levee supporting the proposed I-wall in the Mirabeau breach section is shown in the summary table (A-1) as stable (factor of safety > 1.3) at a canal level of 11.9 ft (NGVD29) without any need to add resistance from the sheet pile. Some resistance from sheet pile was required to attain an acceptable factor of safety for the northern reach of the proposed I-wall (B-2). It is apparent Eustis assumed the head loss is the sand to be greater than would be expected for porous beach sands.

The "factor of safety" concept has been mentioned above, but deserves more attention. The factor of safety applied to the design of earthen embankments or levees is "inseparable from the procedures used to measure shear strengths and analyze stability" (HQUSACE EM 1110-2-1902, p. 1-1). The NOD specified factors of safety generally ranging from 1.25 to 1.5 for sheet pile and levee stability analyses on GNO HPS levees and floodwalls. A factor of safety greater than 1.0 indicates that capacity exceeds demand, while a value less than 1.0 predicts failure of the structure under the design load. USACE guidance states

that "two of the most important considerations that determine appropriate magnitudes for factor of safety are uncertainties in the conditions being analyzed, including shear strengths and consequences of failure or unacceptable performance" (HQUSACE EM 1110-2-1902, p. 1-1). The American Society of Civil Engineers called recently (August 25, 2006) for a

> "full investigation and analyses…to be performed to ensure that the floodwalls and levees have adequate margins of safety against failure, consistent with critical life support structures. Such was not the case pre-Katrina, and may not always be the case now." (http://www.asce.org/files/pdf/Ch9_WhatMustWeDoNext.pdf)

The factors of safety applied to the design of GNO HPS floodwalls and levees appear to be consistent with standards of practice for Corps Engineers levees at the time, although industry would not use a F.5 or less than 1.5 for temporary flooding or 2.0 for long- term flooding. Application of higher factors of safety would have driven up costs. It is likely that, post-Katrina, this standard will change to better incorporate a more rigorous assessment of the "consequences of failure." The London Avenue Canal design history provides a cautionary note, however, that the civil engineering profession should not ignore. The floodwall failures on the London Avenue Canal, and, as we will show later, the 17th Street Canal, might not have been prevented solely by application of higher factors of safety in the design process. Factors of safety are meant to lower natural vulnerabilities and 'constrained' model uncertainties. The 'constrained' means that there are specific references to things like an appropriate method of analysis, an adequate soil investigation, and that standard quality assurance and control methods are used to assure an acceptable follow through on site. Factors of safety cannot be used effectively to manage human and organizational errors. No factor of safety imposed will prevent failure if site-specific conditions are not carefully assessed, and all possible modes of failure adequately addressed.

In 1994, just prior to the start of construction, the contractor that was selected for the London Avenue Canal I-wall project requested permission to degrade 2 feet from the landside of the levee crown of the 17th Street Canal to facilitate construction. The existing levee crown was generally 2 feet higher on the protected (land) side, averaging +4.0 to 5.0 ft, and +2.0 to 3.0 ft on the flood side (Figure 101), so this would have produced a more level crown across the floodwall. Eustis Engineering was asked to assess the effect of such degradation on wall stability for both east and west banks (Eustis Engineering 1994). The analysis indicated that sheet pile wall stability was very sensitive to such a

change, and the factor of safety dropped below the required level anywhere the landside levee was degraded below +3 ft. It is not clear how this advice was received or applied, although it is apparent from the as-built drawings that some excavation was anticipated (Figure 106a). A degrading of this type was allowed on the 17th Street Canal floodwalls, as will be discussed below. The original sheet pile was left in place (Figure 106b).

So, it appears that all analyses required by the USACE were completed for the London Avenue Canal floodwalls. In 1994, an I-wall was constructed on the east bank at Mirabeau rather than a T-wall, and sheet piles were driven to -16 ft (NGVD29) as had been recommended (Eustis Engineering 1993). This wall failed catastrophically during Katrina as all now know (Figure 107).

More of the design record for the London Avenue Canal project has been made available by the Corps of Engineers than for the 17th Street Canal or the IHNC, so we have used it to gain insight into how decisions may have been made elsewhere. It appears that the design resulted in factors of safety that met minimum requirements, although slope stability on some levee reaches was marginal. We have identified one apparent oversight in the choice not to conduct a more rigorous analysis of seepage potential. On the other hand, we have seen no indication that finite-element analysis of seepage was required, requested or applied on any element of the East Bank GNO HPS, despite USACE guidance since the 1980s advising application of these widely available numerical techniques in critical situations. It is instructive to note that T-walls have recently been installed in the Mirabeau breach area as part of the post-Katrina repair, with sheet piles that penetrate the sand layer entirely to act as an effective cut-off.

It is clear that a perceived need to lower cost provided much of the motivation for the design changes made since the 1989 GDM was issued. Changes were justified in the 1990 Revised GDM as necessary to provide some level of "interim" 100-year protection in the near-term. On the other hand, it is apparent that the London Avenue Canal floodwalls were intended to be as permanent as any other feature of the GNO HPS into which they were incorporated. The NOD and LMVD ultimately accepted changes proposed by outside engineers under contract to the Orleans Levee Board that had the effect of reducing reliability.

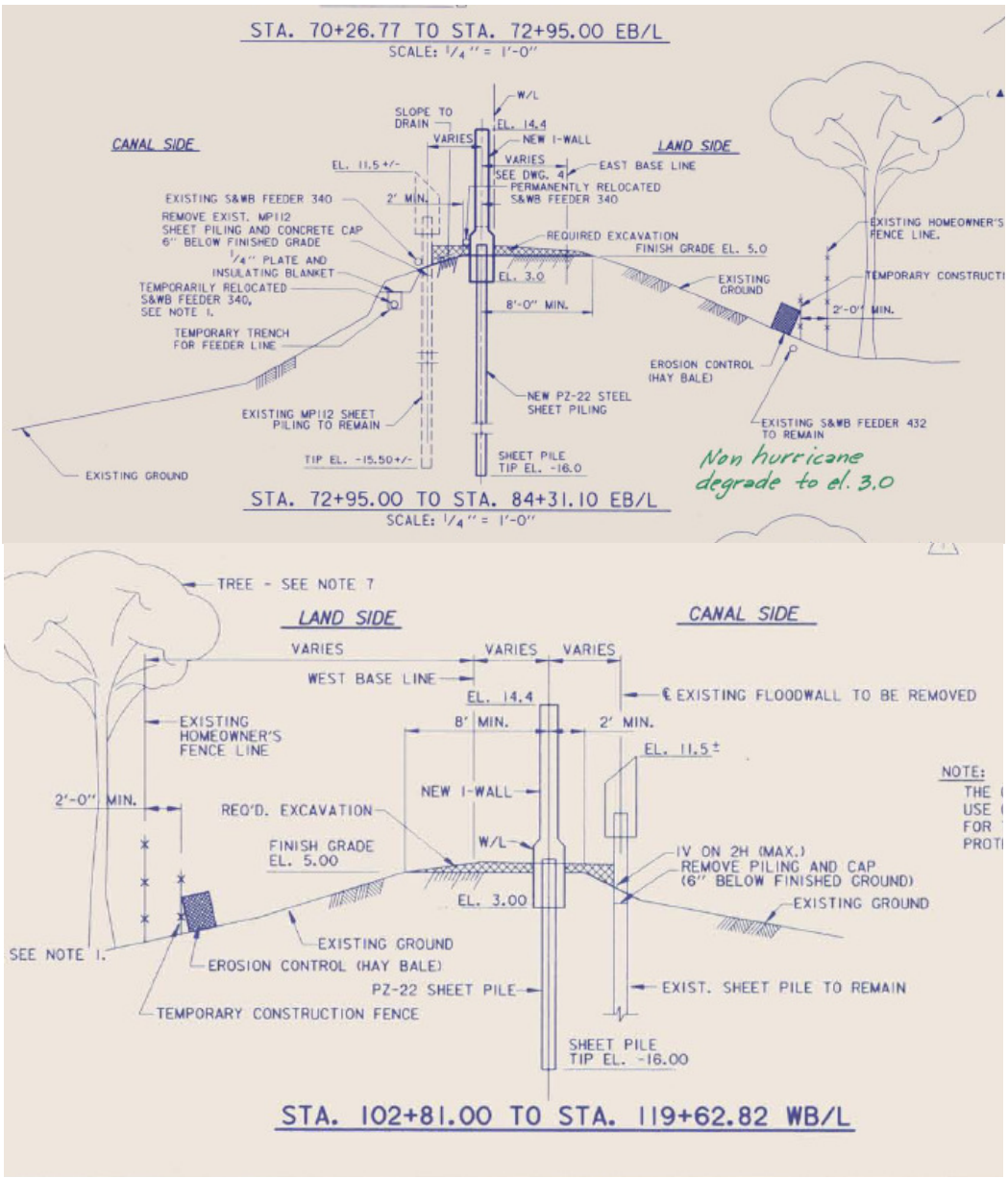


Figure 106a. As-built drawing for London Avenue Canal I-wall in vicinity of south (Mirabeau) breach through east bank (top panel) and north (Robert E. Lee) breach through west bank into Filmore neighborhood (bottom panel).

Figure 106b. Photo showing pre-existing sheet pile in front of I-Wall at Robert E Lee breach at London Avenue Canal.

Figure 107. Efforts to stem flow through the London Avenue Canal east bank breach north of the Mirabeau Avenue bridge. The breach is about 90 ft wide and has swallowed three I-wall monoliths, while adjacent panels are tilting into the hole (IPET 2006b, V-8-14).

Failure Mechanisms for Orleans Metro Floodwalls

We are able to follow the design evolution on the London Avenue Canal project in a way that is not possible for any of the other floodwalls that failed. As-built design drawings for the 17th Street Canal project are not yet part of the public record (https://ipet.wes.army.mil) even though it is over thirteen months since

the catastrophe. The need to quickly close breaches and initiate repairs meant that damaged I-wall panels and sheet piles that should be examined were often buried.

Post-Katrina investigators had to work around a lack of information on sheet pile penetration at the 17th Street Canal breach site, for example, in trying to reconstruct the failure. The 1990 17th Street Canal GDM 20 shows I-wall sheet piling in the vicinity of the breach proposed to an elevation of -10 feet (Figure 108). Non-destructive, in-situ seismic testing of the surviving wall adjacent to the breach were conducted by IPET and Team Louisiana that suggested that the -10 ft GDM value was correct. Sheet piles were pulled from under the surviving I-wall monoliths on either side of the breach in a dramatically staged December 2005 press event (Figure 109).

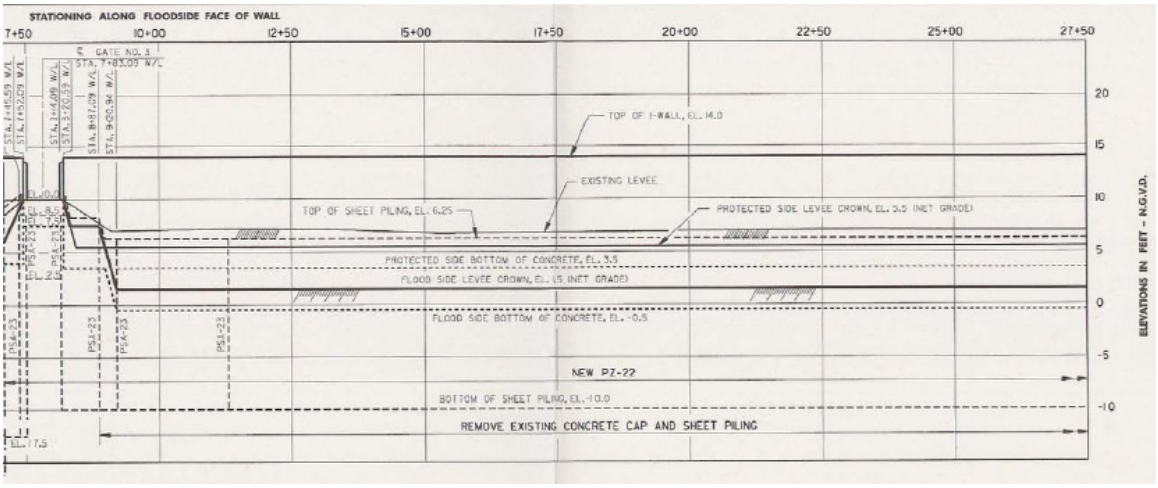


Figure 108. Portion of design drawing from 1990 17th Street Canal GDM 20 showing a -10 ft (NGVD29) tip penetration elevation for I-wall in vicinity of the breach on east bank (Sta. 9+50 to 12+00). Old Hammond Highway bridge is on left. Actual tip penetration elevation was determined to be -17.5 ft (NGVD29) only after surviving I-wall monoliths were demolished and sheet piles were extracted in December 2005.

The piles proved to be more than 23 ft long, indicating a tip penetration elevation of -17, rather than the -10 foot value proposed in the GDM. Correspondence included with the 1990 GDM shows that LMVD had expressed concerns about an inadequate penetration to head ratio (< 3:1). While these queries were initially rebuffed by the NOD, it appears that an adjustment was made later that is not part of the public record. It turned out that while the IPET and Team Louisiana

Figure 109. Sheet pile recovery from south end of 17th Street Canal breach on December 12, 2005.

seismic tests were conducted independently, both teams sent the data to the same individual for analysis, who apparently introduced errors during data reduction that resulted in both teams getting the same initial misleading results (van Heerden and Bryan 2006). All of the confusion could have been avoided, however, by reference to as-built documentation, had it been made available by the NOD (still not available as of this report).

We now proceed to the specific mechanisms of failure identified for the Orleans Metro floodwalls. The design history outlined above for the London Avenue Canal provides a good introduction, so it is logical at this point to extend the discussion through what is known of the breaching processes on this canal first.

Investigation of the London Avenue Canal Breaches

The entirety of the London Avenue Canal is constructed over beach sands of the Pine Island trend, that are encountered at -10 ft, or about 13 to 15 ft below the crowns of the I-wall levees for most of the canal, but are somewhat deeper in the vicinity (station 113 to 118) of the northern breach (Figures 110 and 111). A thin veneer of peaty swamp deposits covers the sand. The thickness of the sand varies somewhat, but generally extends to a depth of -40 or -50 ft (NAVD88). It is not uniform or massive, but contains thin layers, lenses and stringers of fine-grained silts and clays. The levees are composed of variable fill materials. Sheet pilings were driven to tip penetration elevations ranging from -14 to -16 ft on the west side, and from -14 to -28.5 on the east bank. In some places, it appears that

pre-existing piles were used, but the as-built drawings show that new sheet piles were used in the breached sections (Figures 97 and 98).

The London Avenue Canal is shallower than the 17th Street Canal, with an average invert elevation of -10 feet (NAVD88), but this is sufficient to ensure a hydraulic connection with the beach sand (Figure 112). The canal is perched to some degree above the surrounding neighborhoods that have a ground elevation that slopes from about -4 ft at Mirabeau Avenue to about -7 ft in the Filmore area. ILIT (2006) points out that because the canal has never been drained in more than a century, the peaty layer above the sand in the immediate vicinity of the canal has not experienced the same degree of settlement caused by oxidation and consolidation as has occurred in the adjacent protected areas (Figure 112).

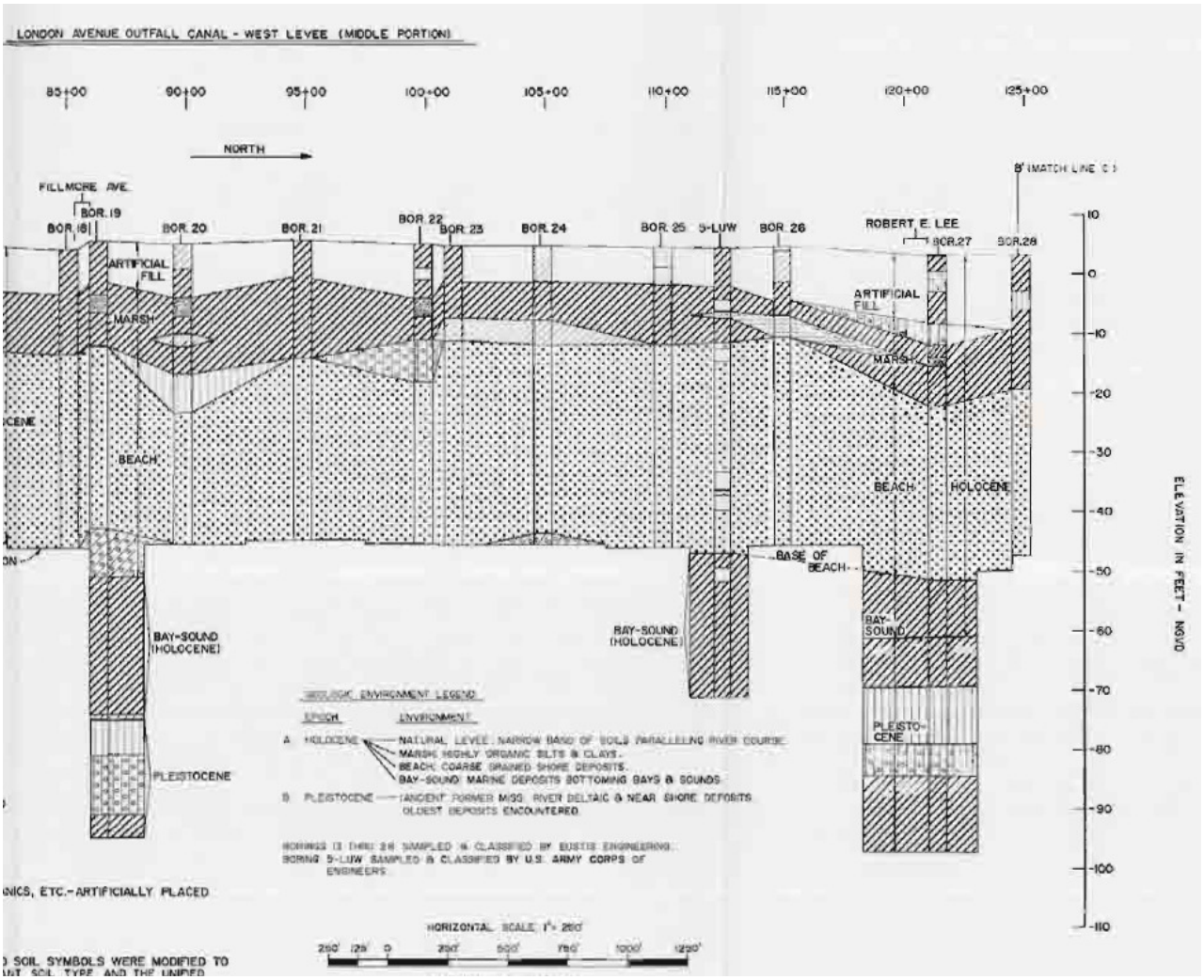


Figure 110. Geologic profile from borings along the west levee alignment of the London Avenue Canal on the north end. Failure zone of the Robert E Lee (north) breach extends from Station 113 to 118 (GDM 19A).

London Avenue South Breach at Mirabeau

About 90 ft of floodwall failed at this location (Station 75), making this the narrowest of the drainage canal breaches. It was also the deepest, however, as the east bank levee was completely removed to the -4 foot elevation of the surrounding neighborhood, and a large hole formed under the landside slope of the levee (Figure 113).

A large volume of beach sand was eroded from the bed of the canal, deepening it by 20 feet adjacent to the breach (Figure 112). This sand was deposited in large dunes in a fan radiating into the Mirabeau subdivision (Figure 114). This was the only drainage canal breach that does not appear to have involved any significant lateral translation of the I-wall, as the panels under the breach location dropped straight down into the hole created by the erosion (Figure 107).

The ILIT (2006, p. 8-34) conducted both finite-element and flow net analyses of the Mirabeau breach, varying the horizontal and vertical permeabilities in the marsh and sand strata (Figure 115). Both of these methods were available to engineers in the 1980s and 1990s. ILIT concluded that for all reasonable ranges of these parameters, "the soils in the inboard (landside) toe area were vulnerable to erosion and potential piping at storm surge levels of less than +9 feet MSL." Indeed, IPET investigators initially had difficulty delaying initiation of failure in their models until the canal got that high, and suggested that a thin layer of fine-grained sediments at the bottom of the canal may have initially inhibited the pressure transmission. IPET now believes the Mirabeau breach started at a canal level of +7 feet (NAVD88), and was the first breach on the London Avenue Canal.

Team Louisiana geotechnical engineers with many years of experience prior to the advent of sophisticated computer techniques believe that a conservative assumption for this site should have been that the water pressure in the canal would equal that at the landside toe of the levee, and as shown previously, a simple calculation of the weight of the overburden clay would reveal that it was insufficient to prevent piping or sand boil formation. They complain that this fundamental assumption was never adequately tested or addressed because of slavish reliance on the creep ratio method, which incorporates empirical factors that are inappropriate for the site. This reliance on a 'cookbook' approach appears many times in the history of what led to the Katrina flooding catastrophe.

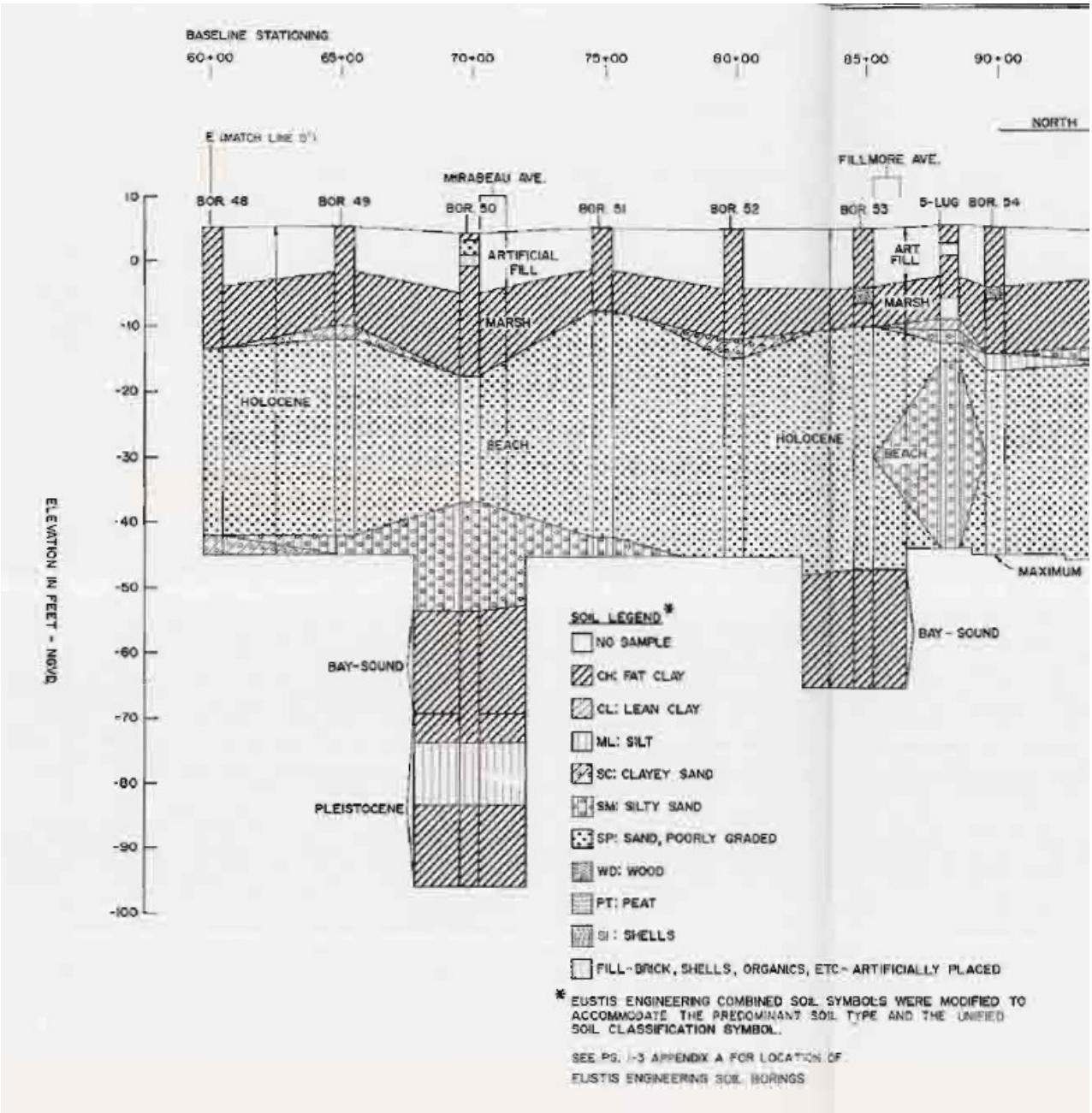


Figure 111. Geologic profile from borings along the east levee alignment of the London Avenue farther south Mirabeau (south) breach is at Station 75 (GDM 19A).

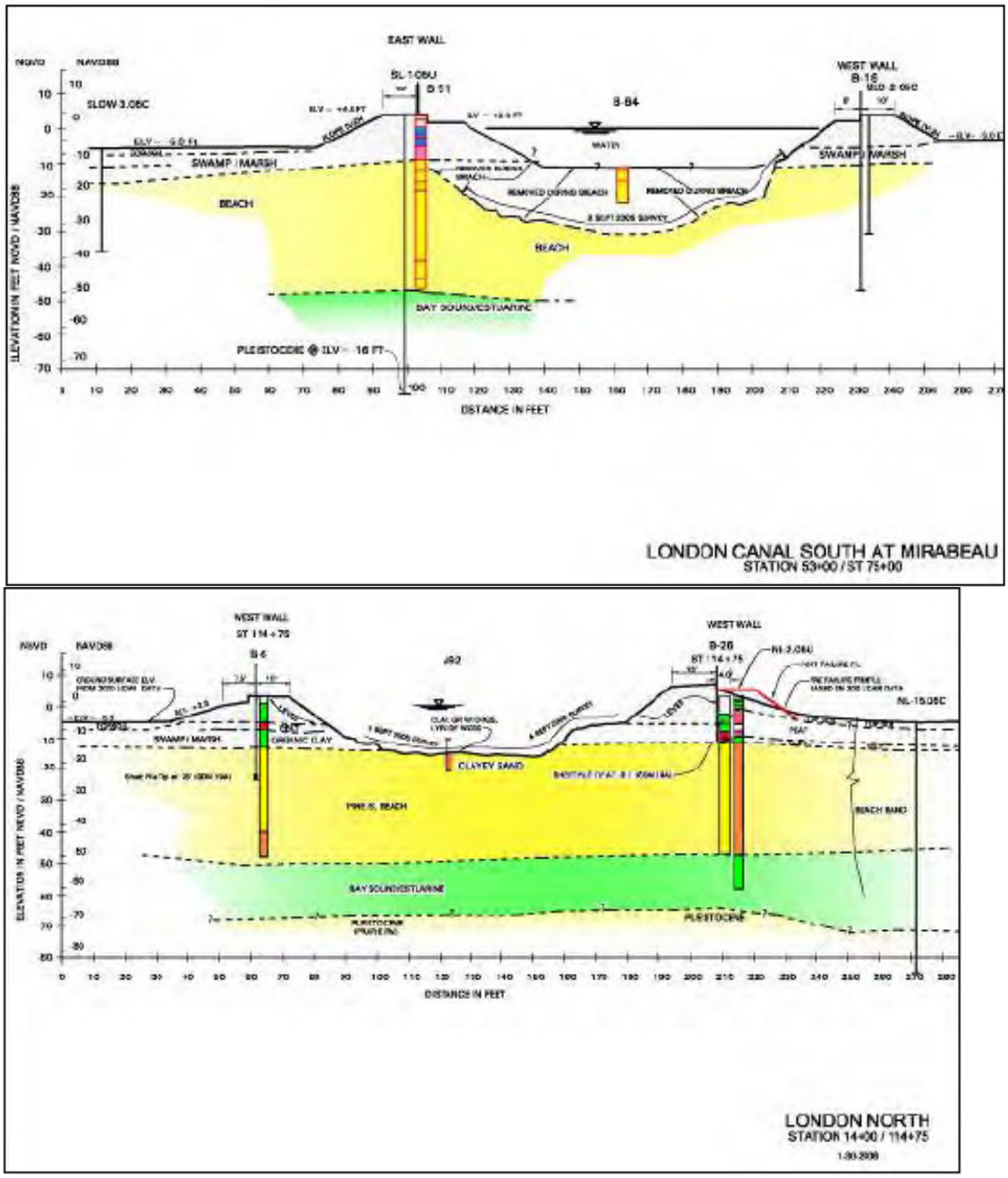


Figure 112. Comparison of London Avenue Canal cross-sections at the two breach locations looking upstream, showing erosion in canal after breaching, and pre- and post-Katrina levee profiles. Breach is left through east levee at London South (Mirabeau), and right through west levee at London North (Robert E. Lee) (IPET 2006b, V-43, V-44).

Figure 113. View north across London Canal south breach at Mirabeau, showing outlet channel and deep scour hole.

Figure 114. Sand dunes deposited in Mirabeau subdivision by discharge from the London Canal North breach. Note clay blocks uplifted and pushed aside by discharge in foreground.

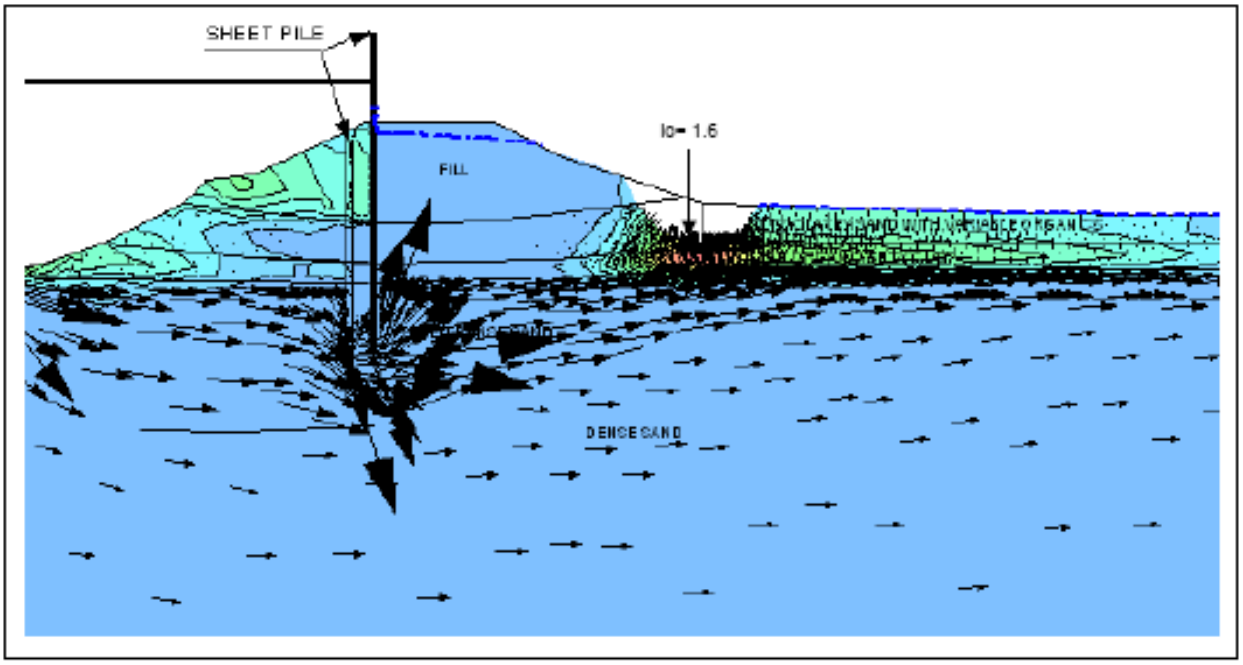


Figure 115. Results of ILIT (2006, 8-125) flow net analysis for a canal water level of +9 ft (NAVD88) showing flow velocities under the levee and sheet pile, and breaching of the marsh deposit on the landside toe of the levee.

ILIT (2006, p. 8-34) further concludes that the sheet piles at Mirabeau "were nowhere near deep enough to be effective in reducing the massive underseepage flows through the pervious sands." Because the canal is perched above the surrounding neighborhood, uplift pressure developed in the sand was transmitted under the sheet pile until it could not be resisted by the thin peaty overburden at some point on the inboard levee slope. Once the overburden was breached, a classic sand boil developed that provided a short outlet drainage path (Figure 115). The boil attracted additional discharge and velocities increased sufficiently to erode and transport sand from a formation that was already weakened by the increased pressure. Removal of sand ultimately created voids beneath the levee that led to failure of the I-wall it supported. As the breach widened, velocity decreased. The breach cross-section stabilized when the velocity dropped below that necessary to transport sand, although discharge of clear water continued into December when a sheet pile cofferdam was completed.

The failure at Mirabeau is perhaps the simplest to explain. Yet this I-wall had a factor of safety for sheet pile stability of more than 2.0 because it was driven into dense sand (Eustis Engineering 1994). **The design deficiency arose, as has been discussed, in the failure to conduct a rigorous seepage analysis.** As ILIT has

shown, vertical flow velocities that developed when the canal rose may have been as much as an order of magnitude higher than the horizontal velocities in the vicinity of the sheet pile (Figure 115), and much greater than would have been predicted by an empirical approach like Lane's Weighted Creep Ratio method employed to check for seepage potential by Eustis Engineering (1993).

London Avenue North breach at Robert E. Lee Boulevard

About 410 ft of floodwall failed at this location (Station 118). The as-built drawings indicate that the I-wall was constructed with a crown elevation of 14.4 ft (NGVD29), and the sheet pile was driven to a tip penetration elevation of -16 feet (Figure 106).

The soil layering under this levee is somewhat more complex than at Mirabeau in that the top of the sand dips toward the lake, and is from 5 to 10 ft deeper across the breach site, so that more of a marsh overburden is present (Figure 110). The surficial marsh/swamp deposit appears to be divided into two layers separated by what ILIT (2006, p. 3-15) has called a "highly plastic and water-rich organic clay," no more than 2 to 3 ft thick, that is shown in the GDM 19A west bank section across the breached reach, but not extending much farther north or south (Figure 110). This clay is continuous across the canal as it appears in the GDM 19A geologic section on the opposite, east bank, at the interface of the marsh and sand beneath the distressed wall section (Figure 104). The Pine Island sand extends to a depth of about 45 ft, below which is a blue-grey clay indicative of deposition in a lagoon or bay.

The west bank levee contained sheet piles associated with an older structure that had been cut at grade, but left in place on the canal side of the levee when the current I-wall was built (Figure 106). When the breach occurred, this line of sheet pile formed a seam (Figure 116). The outboard portion of the levee largely stayed in position, while the remainder of the levee was pushed about 20 feet landward creating a gap. As the inboard side of the levee containing the I-walls slid, the wall panels rotated landward (Figure 117). A thrust block associated with the slide raised some backyards in the middle portion of the breach, including an intact outbuilding that the owner has called a "playhouse" (Figure 118).

Figure 116. View of south end of London Avenue Canal north breach showing older sheet pile on left retaining the inboard portion of the west bank levee, while the 1994 I-wall on right has translated and rotated landward.

The length of the breach and the leaning wall panels are suggestive of a deep-seated slope stability failure or, possibly, one caused by inadequate sheet pile penetration, but it appears that seepage also played an important initiating role. A very large sand boil crater was observed at the position of the landside levee toe near the north end of the breach that was later covered by road construction (Figure 119). ILIT observed a similar, but smaller feature, farther south in the vicinity of the raised playhouse (ILIT 2006, p. 8-130).

Sinkholes were observed at the base of the distressed wall section on the other side (east bank) of the canal, and sand boil features were also noted at the landside toe of that levee. Accordingly, it appears to us that seepage at the north end of the west side breach probably provided a precipitating mechanism that fed into a more general slope failure. Both IPET and ILIT believe that a gap formed on the canal side, and certainly this happened at some point, but if the underseepage was great enough it may not have been necessary to explain the failure. It is also possible that the gap may have occurred simultaneously with

the slope failure and contributed to the overall length of the breach. With so many failure mechanisms at work, the sequence will probably never be known conclusively.

Figure 117. Translation of I-wall landward seen from south end of London Avenue north breach into Filmore neighborhood, looking north. Old sheet pile wall has remained in place and retained the inboard levee slope while the I-wall and remainder of levee has moved.

One indication that seepage was probably important at the beginning of the London Avenue Canal north breach is the volume of beach sand deposited near the north end of the breach (Figure 119). Such transport is indicative of a high velocity discharge through an initially narrow breach that was reduced as the bank failed progressively to the south (Figure 120). This sequence is also suggested by the lesser erosion of sand from the canal bed that occurred at this location, compared to that in the vicinity of the Mirabeau breach (Figure 112).

IPET now believes this breach began when the canal level was at about +8 ft (NAVD88), shortly after the breach at Mirabeau.

Figure 118. View looking south at north end of London Avenue north breach into Filmore neighborhood showing landside displacement and rotation of I-wall panels. "Playhouse" in background to right behind tree was heaved up on a thrust block and moved landward as the levee slid.

Figure 119. View of large sand boil feature, still seeping two weeks after the breach, looking northwest at position of levee toe near north end of London Avenue Canal north breach. Note the deposits of beach sand drifted against the walls of the houses and clay blocks lifted by water.

Figure 120. Fully developed north breach on London Avenue Canal. Sand boil features were observed at north (left) end of breached section (ILIT 2006, p. 8-129). In the center of the picture can be seen two trees that have fallen in to the canal, no doubt by the force of the wind as the roof systems were weakened by the breach.

Investigation of the 17th Street Canal Breach

This breach is possibly the best known floodwall breach in history, given the amount of press coverage after Katrina. About 450 feet of the east bank floodwall south of the Old Hammond Highway bridge (new station 7+50 to 12+50, or old station 560.25 to 565.25) failed in a way that appears to have been more sudden and violent than was true for the London Avenue Canal breaches (Figure 121). The breach had two outlets separated by a 200-foot middle section in which the landside of the levee slid forty feet but remained intact. The southern outlet appears to have carried a more violent flow. Houses and small apartment buildings in the throat of the south outlet were leveled, while homes adjacent to the north outlet sustained damage similar to what was seen at the London Avenue Canal breaches (Figure 122).

As was true for the London Avenue Canal, the 17th Street Canal was originally dredged in the late 1800's to facilitate drainage from the city, initially through gravity flow, and after 1898 by pumps. Material excavated from the canal was placed along the banks to form a low levee. As development spread toward the

lake, earthen levees were raised to reach the current configuration by 1962. An earlier I-wall was added in 1971. In 1989, the Orleans Levee Board began to drive sheet pile into the levee with a top elevation of +10 feet to provide interim hurricane surge protection and to accommodate increased discharge associated with installation of new pumps by the Sewerage and Water Board at Pump Station 6, located at the southern terminus of the canal. The 1990 GDM 20 was developed at this time to increase protection against surge to the SPH level and to ensure levee stability after the canal was enlarged and deepened to accommodate the increased discharge. The original sheet piles were then driven to about a -17 ft (NGVD29) elevation and capped with concrete monoliths in 1994.

Figure 121. FEMA photo of the 17th Street Canal breach around 2 pm on August 29. Jefferson side (foreground) was also flooded to a lesser depth by flow around the south end of the canal from Orleans Parish. Note north (left) and south (right) outlets to breach. West bank levee is bounded by public roadway, while east bank ROW abuts private property.

At the time of I-wall construction, the original sheet piling was driven down until even with the top of the earthen embankment. The next step was to be the attachment of the concrete monoliths. However, because of an approaching tropical storm, the pilings were pulled back up to their original elevations. Once the storm threat was over the pilings were then pushed back into the soft soil. These actions of push, pull, and then push weakened the soils and created

Figure 122. View of Lakeview neighborhood through south outlet of 17th Street Canal breach, showing scour of swamp stumps and complete removal of buildings from pile supported foundations. I-wall monoliths rotated landward and were flattened by the discharge.

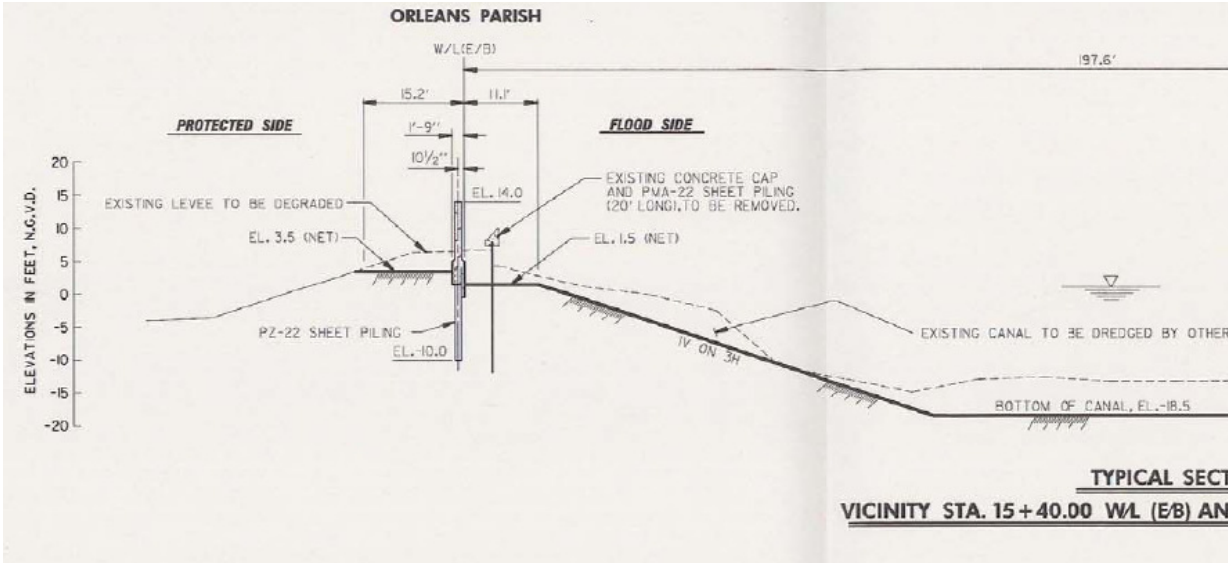


Figure 123. Typical 17th Street Canal east bank section for reach including the breach showing work proposed in 1990 GDM 20. Sheet pile was actually installed to a tip penetration elevation of -17.5 ft, rather than -10.0 ft as shown here. Note degrading of existing levee on canal side to approximately mean sea level (+1.5 ft NGVD29) to accommodate 1V on 3H canal-side slope to -18.5 invert elevation, as well as degrading of protected side to +3.5 ft (NGVD29). I-wall proposed includes an unbalanced monolith with a longer leg on the canal side.

problems for the contractor selected to cast the concrete I-wall cap. The contractor, Pittman Construction Company, Inc., had difficulty getting the particular concrete monoliths cast in this section to stay within the required tolerances for the top of wall alignment.

The monoliths to be built in this section were of an "unbalanced" type, in that the leg of the cap on the canal side was 3 to 4 ft longer than on the protected side, and the contractor complained that the weak soil strengths made it impossible to control movement of the sheet pile and the form during the casting process (Figure 123). The out-of-tolerance I-walls were eventually accepted by the USACE, but the contractor was not compensated for the increased time and expense.

While this controversy would otherwise be unremarkable, engineers consulting with Team Louisiana were puzzled by the specification of such a monolith. No rationale is provided in GDM 20. It is now believed that the eccentric I-wall monoliths and the degrading of the levee crown were connected to a similar concern about levee stability into the canal. The longer foot on the canal side might add stability to the I-wall while allowing for a stable 1V to 3H slope into the deepened and widened canal bottom without increasing the mass of the east bank levee or requiring that the sheet piles already in place be set back (Figure 123).

A cross-section of the canal and adjacent levees at the Old Hammond Highway crossing in GDM 20 shows that the center line of the canal is not equidistant between the two levees, but is closer to the east side, making that bank steeper than on the west (Jefferson Parish) side (Figure 124). This is supported by the observation that the east bank inside the I-wall does not support vegetation, while that on the Jefferson Parish (west) side does.

As was noted previously, the sheet piling in the breached section was actually driven to about -17 ft (NGVD29), rather than the -10.0 ft given in GDM 20 (Figure 123). It is not clear why this change was made, but the deeper penetration did not lower the sheet pile below the deepened canal invert (-18 ft), creating at least the possibility of seepage. Indeed, adjacent property owners had noted and reported to various authorities that their back yards were quite wet even after long periods of dry weather, as might be expected from a canal seep.

The geology underlying the levee that breached at 17th Street Canal differs significantly from that described for the London Avenue Canal primarily

because the Pine Island sands are much thinner in section and are located at a depth below -35 ft (Figure 124 and 125). In the area of the breach, the east levee is constructed primarily on fat or soft clays, except for about 5 ft of marsh/swamp soil classified as a peat. At the south end of the breach site, the peat is overlain by a few feet of clay, while at the north end the levee fill sits directly on the peat (Figure 125).

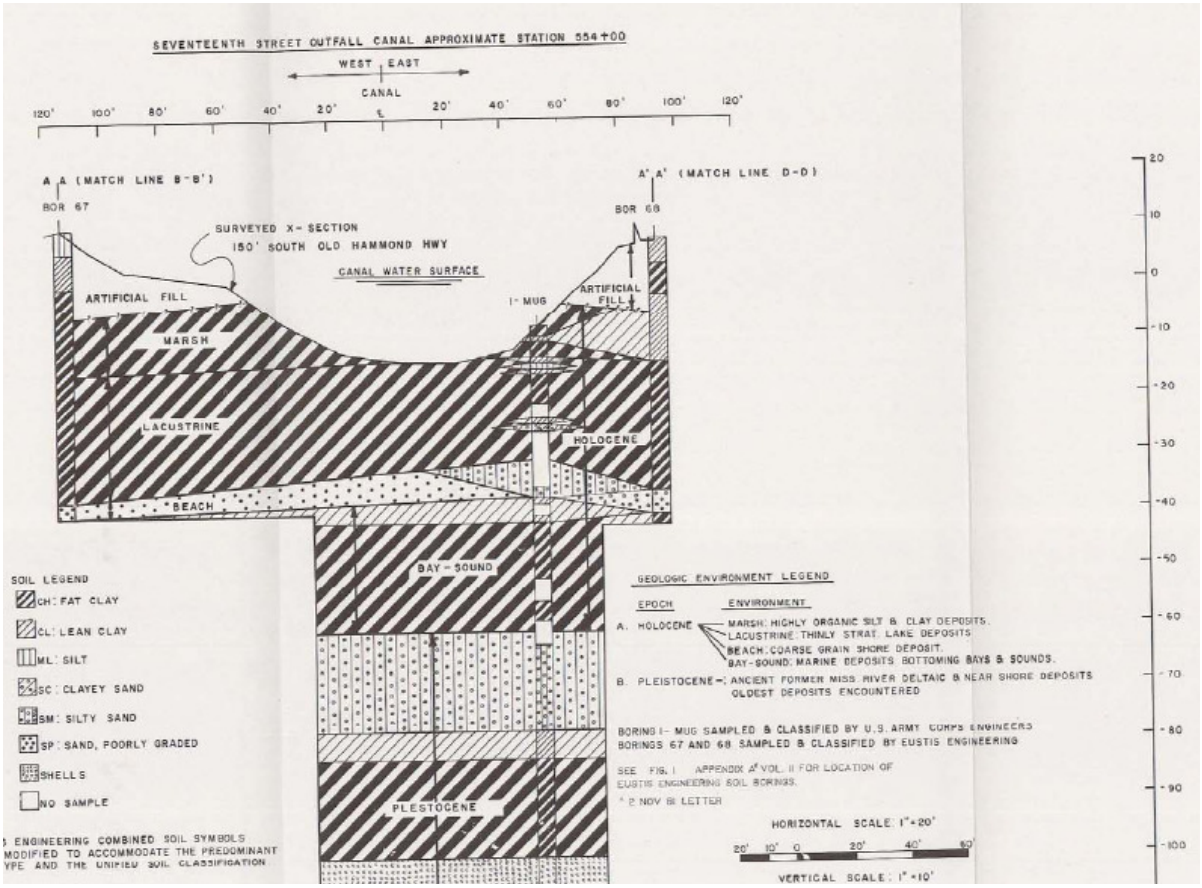


Figure 124. Cross-section of 17th Street Canal between the breach location and the Old Hammond Highway bridge shows that canal centerline is closer to the east bank (right) than to the west (left), making the bank slope on this side steeper.

During Katrina, a 200-foot section of the east levee slid sideways 40 to 50 feet in a classic translational slide failure that caused little disturbance of the chain-link fence constructed atop the surviving landside half of the original levee (Figure 126). The backyards of residents in the path of the sliding levee were heaved up and a formerly flat terrain was transformed into an accordion of ridges as homes, cars and trees were 'bulldozed' into strangely skewed positions (Figure 127).

A detailed cross-section in the vicinity of the breach based on borings acquired by ILIT and Team Louisiana engineers shows the levee fill sitting on a highly organic swamp soil stratum with a thin, soft clay layer at its base between -12 and -16 ft. The sheet pile just barely penetrates through the swamp peat (Figure 128). ILIT has described a transition or intermixing zone about 4 ft thick beneath the marsh deposits that gradually becomes less organic as it grades into a soft, grey, lacustrine clay. ILIT has presented evidence that the translational slide that carried away the I-wall at this breach moved on a slip surface developed in the thin clay layer at the base of the swamp/marsh deposits.

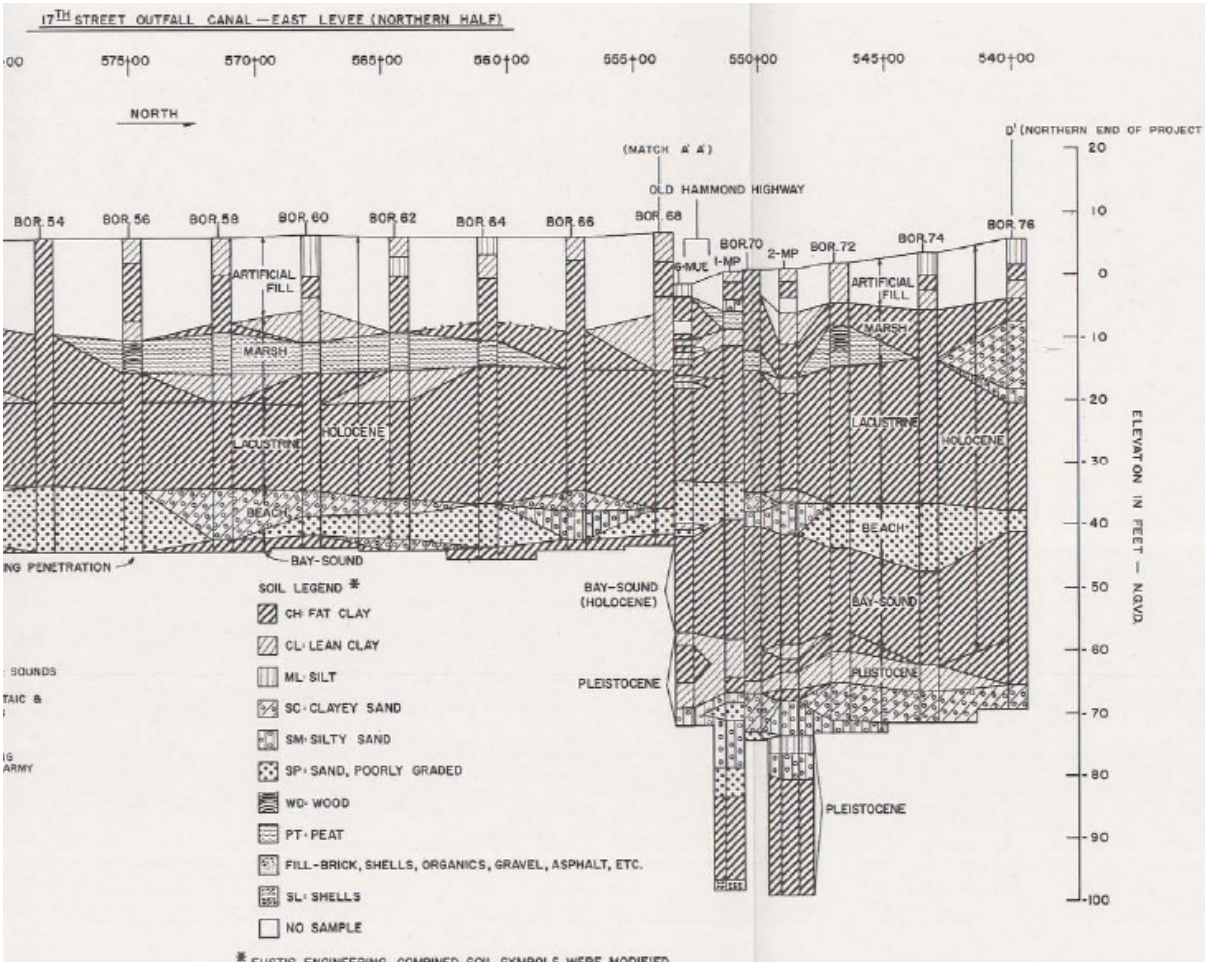


Figure 125. GDM 20 geological section along east bank of 17th Street Canal. Breach location is between stations 560 and 565.

Borings were acquired in 1981 for the canal deepening project by the Sewerage and Water Board of New Orleans (SWBNO), and were incorporated into GDM 20. The east bank canal floodwall was designed in several reaches, but soil strength properties were averaged over an 8,200 foot reach, extending from station 553 to 635 (Figure 129). **Given the complex geology of this area, and the**

problems presented by these soils for construction, Team Louisiana engineers point out that this was too great a distance for averaging critical properties such as soil type and strength that also varied within depth zones.

Figure 126. View of surviving section of east levee of 17th Street Canal that moved to the left between 40 and 50 feet carrying the fence with it. The concrete monoliths that formed the I-wall in this section have fallen toward the canal with tops emerging out of the gravel shoulder of the road constructed across the breach. The undamaged wall is visible behind track hoe in the distance.

Figure 127. Heaved house, cars and bus in front of slide block at 17th Street Canal breach.

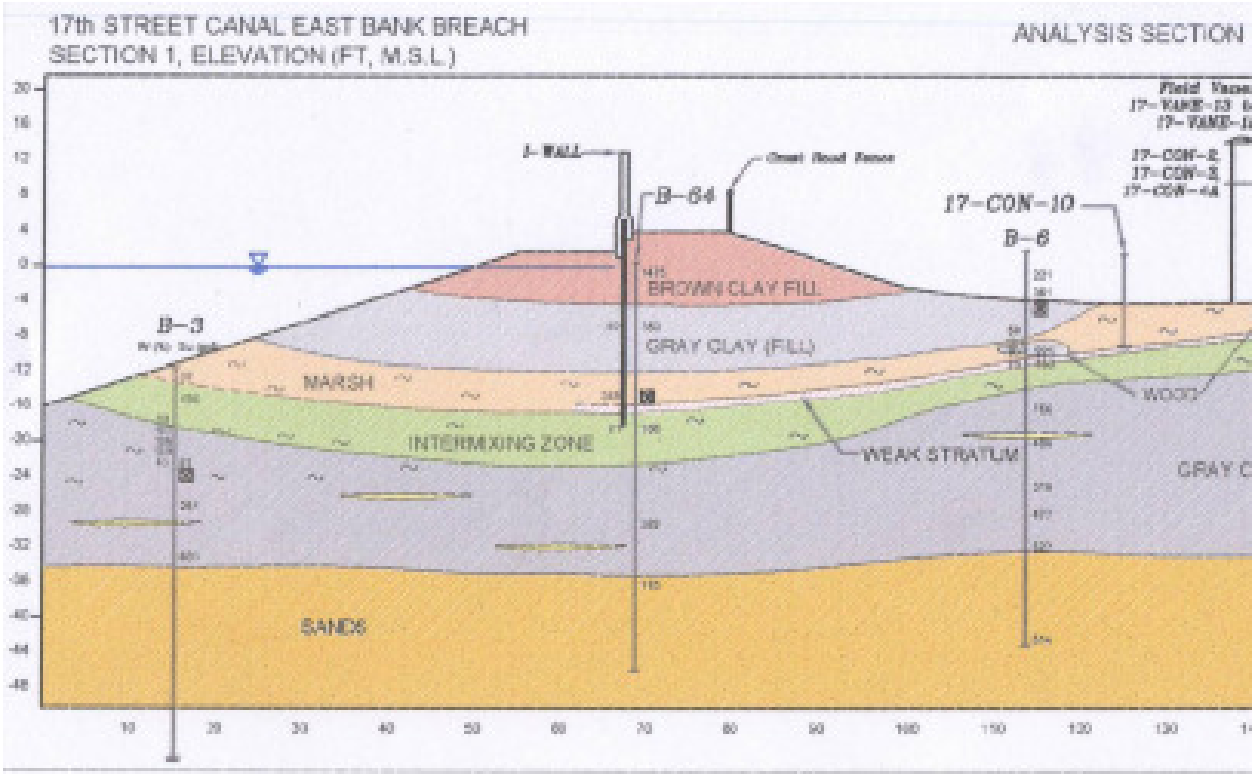


Figure 128. Post-storm ILIT assessment of strata beneath the 17th Street Canal east bank I-wall in the breach zone (ILIT 2006, p. 8-62).

The strength-depth relationship used in the design of the levee and I-wall on the east side of the canal is derived from a series of boreholes drilled primarily on top of 17th Street canal levee but with some borings from the canal and at the toe (Figure 129). Below the drained levee fill, soil strengths in the fat clay (CH) generally range from 0.1 to 0.25 tons per square foot (200 to 500 psf).

The USACE has plotted characteristic values to be used for design. This line shows strength values decreasing from 500 psf at the top of the saturated section (-2.0 ft NGVD29) to a low of 280 psf that characterizes the section down to -10.5 ft. This is about the depth that ILIT interprets as the bottom of a thick sequence of clay levee fill (Figure 128). The characteristic strength line is shown as increasing linearly below -10.5 to reach 360 psf at -20 feet (NGVD29).

Engineers with Team Louisiana saw potential problems with the plotted characteristic soil shear strengths. First, as has already been noted, the length of the reach characterized meant that important outliers indicative of a weak layer or a discontinuous stratum like the peat could have been missed. The entire section down to the sand contact below -36 ft is shown as fat clay (CH), with no notation for the peaty clay (Figure 129). Second, a significant number of the

samples from -5 to -20 ft are plotted at 0.1 tons per square foot (200 psf), well to the left of the characteristic line and 30 percent weaker than the 280 psf minimum of the line.

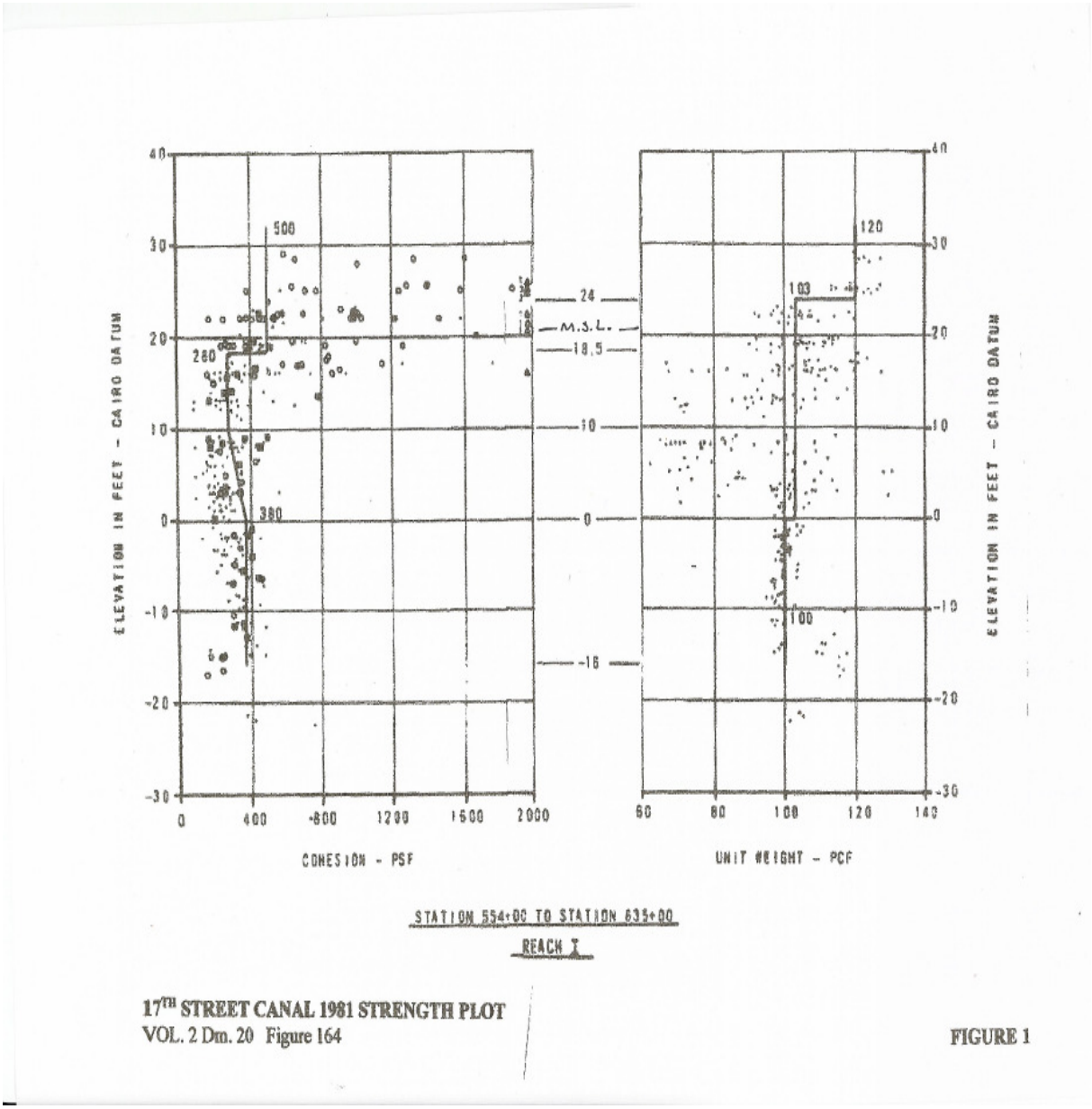


Figure129a. The strength-depth relationship used in the design of the levee and I-wall on the east side of the 17th Street canal, by the Corps in 1981 as reported in Design Memorandum 20 Vol. 2. (Based on Cairo Datum which is 20.5 feet above mean sea level). Note how the assumed strength line misses or ignores all the weak soils between -3 feet and -20 feet below MSL (Cairo datum =18 to 0).

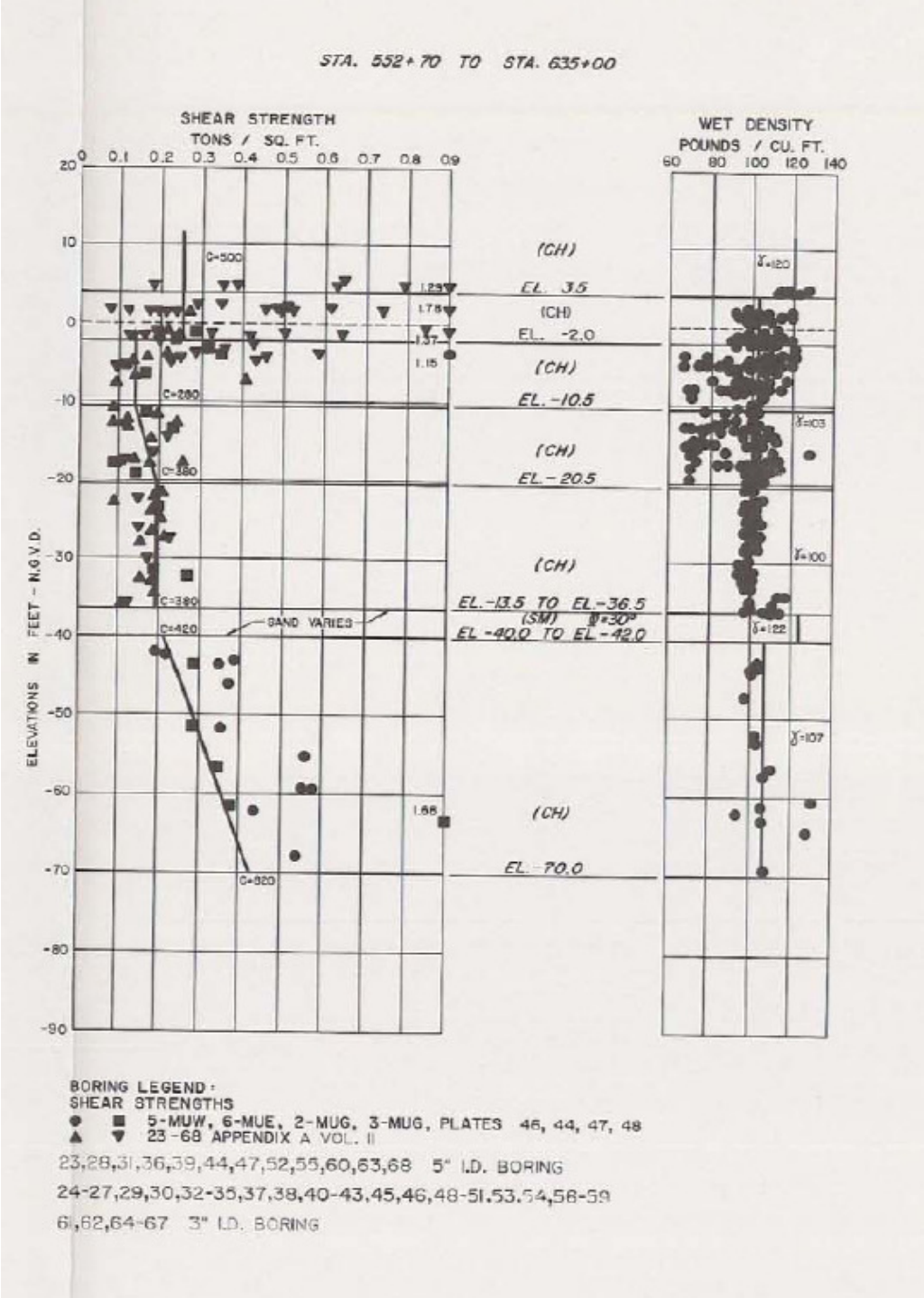


Figure 129b. The strength-depth relationship used in the design of the levee and I-wall on the east side of the 17th Street canal. Line indicates "characteristic" values assigned by the USACE for use in foundation design for cohesive strength on left, and unit weight on right. Zone of most deviation from measured to assigned strength values is from -10 to -20 ft (NGVD29) on left plot. The wet density data (on right) shows very low density soils between -2 and – 20, indicative of organic clays and peat. OH and PT not shown as soil classification between shear strength and wet density plots.

Data are available in GDM 20 for boreholes located in the vicinity of the breach. We plotted a new strength-depth relationship based on nine borings covering the breach zone from station 555 to 568 (1,300 ft) that have been available since 1981 (Figure 130).

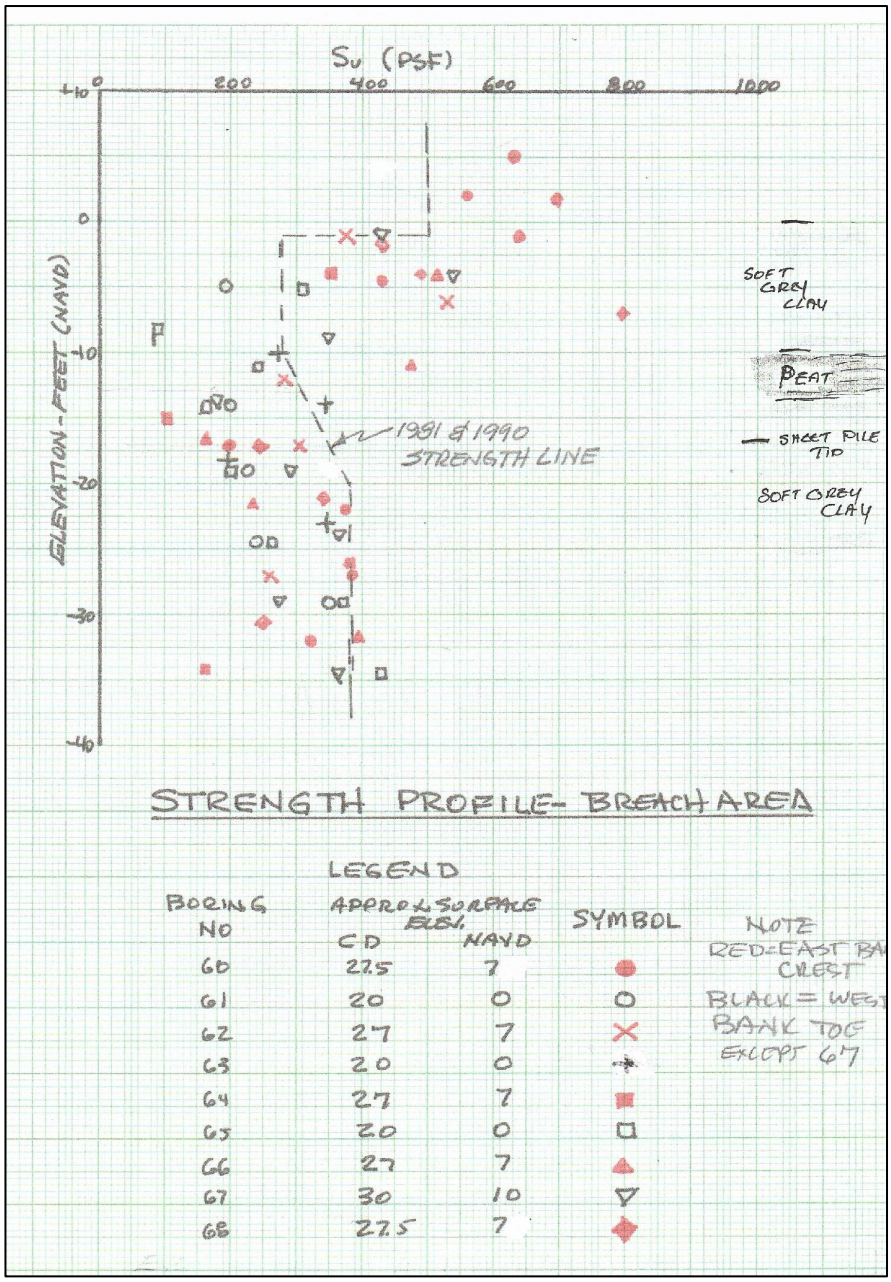


Figure 130. Cohesive strength/depth plot made by Team Louisiana for soils of the east bank levee that failed on the 17th Street Canal using 1981 SWBNO borings (black symbols), including toe borings from the west bank levee (red symbols). Reach characterized extends from station 555 to 568, or 1,300 ft centered on the breach area. GDM 20 characteristic line is shown and overestimates strength for almost all samples from -13 to -19 feet.

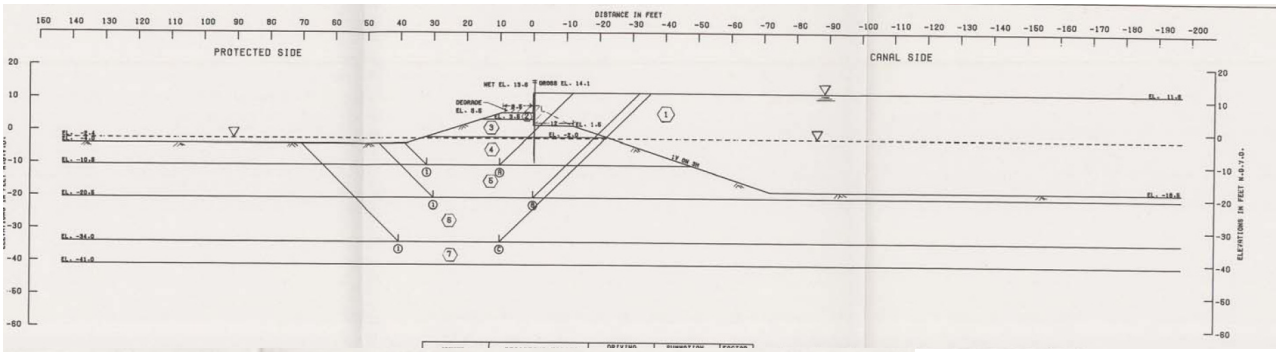
It had been noted that few of the samples plotted on the GDM 20 plot were from borings acquired at the toe of the east levee, perhaps because of access problems, but comparable borings were available from the toe of the west bank on the Jefferson side, where the right-of-way is wider. Cone penetrometer tests (CPT) made by Team Louisiana engineers on the east bank toe after Katrina showed that west bank toe samples should give a good approximation of shear stress in east bank toe soils unconsolidated by the levee load. This is important for slope stability analyses, as will be discussed.

It can be seen that strengths determined on samples from the west bank toe borings converge on those from the east bank crown borings below -10 ft and are comparable below this point, showing little evidence of an increase in strength below the levee fill due to loading (Figure 130). What is more remarkable is that virtually all of the samples below -10 ft (NGVD29) fall below the characteristic strength line from GDM 20. A particularly weak zone appears to occur between -12 and -16 where some values are as low as 100 psf, and average about 200 psf, while the GDM characteristic line shows an average of more than 300 psf should be expected. The borings that yielded these samples show this zone of weakness is associated with the organic-rich soft grey clay that has been interpreted as a swamp/marsh stratum (Figure 128). A layer as thick as 5 or 6 ft occurs in some borings, but this material can be found anywhere from -9 to -21 ft. This variability presumably contributes to the spread of strength values in this depth range, even for a reach of canal levee only 1,300 ft long.

Based on a review of the borehole data, the USACE directed the consulting design engineers to re-analyze slope stability in the failure area in 1989 (Eustis Engineering, Inc. 1989). They used only the east bank crown borings; the same shear strength line as used in 1981 even through they had newer borings (see Figures 129b and 130). In this review only slight changes were made in the geometry such as a reflecting a patio excavated into the levee toe. Eustis determined that the east bank levee in the breach area should be stable, with a factor of safety of more than 1.3 against failure into the canal or towards the land that was unchanged from that documented in GDM 20 (Figure 131). Eustis calculated the lowest factor of safety for a slide toward the landside at -20.4 ft (NGVD29).

We performed the same analysis using the Method of Generalized Slices, available since the 1950's, which produces very comparable results to the Method of Planes. We divided the soils into 13 strength categories using the strength-depth profile we had developed for the area around the breach, and applied

hydrostatic forces generated by a storm surge at elevation +10.8 for the stability analysis. The section fails in a normal slope stability pattern (Figure 132). The calculated critical surface passes through the materials under the levee at about the depth predicted by the NOD contractor, -20 feet, not the weaker soils on the canal side. This work shows that if the soils had been properly analyzed by the NOD, they would almost certainly have found the mass of the levee supporting the I-wall to be too small to resist the lateral force exerted by the elevated canal water, and would have modified the design. The fault was not so much in the analytical approach, but in **an inaccurate assessment of underlying soil strengths. The slavish following of a specific geometry for the method of planes was another contributor; rather than letting the soil characteristics define the critical geometry.**



STRATUM NO.	SOIL TYPE	EFFECTIVE UNIT WT. P.C.F. VERT. 1	EFFECTIVE UNIT WT. P.C.F. VERT. 2	C = UNIT COHESION - P.S.F. CENTER OF STRATUM VERT. 1	CENTER OF STRATUM VERT. 2	BOTTOM OF STRATUM VERT. 1	BOTTOM OF STRATUM VERT. 2	FRICTION ANGLE DEGREES
1	(WATER)	62.5	62.5	0.0	0.0	0.0	0.0	0.0
2	(CH)	120.0	120.0	500.0	500.0	500.0	500.0	0.0
3	(CH)	103.0	103.0	500.0	500.0	500.0	500.0	0.0
4	(CH)	103.0	103.0	280.0	280.0	280.0	280.0	0.0
5	(CH)	103.0	103.0	330.0	330.0	380.0	380.0	0.0
6	(CH)	102.0	102.0	380.0	380.0	380.0	380.0	0.0
7	(SM)	122.0	122.0	0.0	0.0	0.0	0.0	30.0

ASSUMED FAILURE SURFACE NO.	ASSUMED FAILURE SURFACE ELEV.	RESISTING FORCES R_a	RESISTING FORCES R_s	RESISTING FORCES R_p	DRIVING FORCES D_a	DRIVING FORCES $-D_p$	SUMMATION OF FORCES RESISTING	SUMMATION OF FORCES DRIVING	FACTOR OF SAFETY
(A) (1)	-10.5	8409	6180	3738	16348	2921	18327	13427	1.36
(B) (1)	-20.5	12362	11400	10240	41541	15328	34002	26213	1.30
(C) (1)	-34.0	21747	11580	20600	57684	46278	53837	41406	1.30

Figure 131. Protected side slope-stability analysis (Method of Planes) for east bank of the 17th Street Canal (top panel) in the reach including the breach (Station 553 to 568), showing factors of safety of 1.3 or greater (bottom panel) for landside failure at a canal elevation of 11.6 ft (NGVD29).

Cohesive strength values used for slope stability analysis (middle) are not below 280 psf (GDM 20, 1990).

While it was not difficult to predict failure at a canal water elevation of +10.8 ft using methods available since the 1960s, the IPET reconstruction of the failure indicated that it actually happened at a lower canal elevation, perhaps as low as

+6.5 ft (NAVD88). One IPET report suggested that a previously unknown failure mode had been discovered in the process of trying to reconstruct the 17th Street Canal breach (IPET 2006a). However, this turned out not to be the case. In fact, the USACE LMVD was sufficiently concerned about the potential for I-wall failure that they initiated a research program on the behavior of cantilevered sheet pile walls founded in soft soils in the 1982 to 1989 period. This work, called the E99 program, included in 1985 a full-scale test in which a sheet pile wall was loaded to failure at a site in the Atchafalaya Basin with soils similar to those under the Orleans Metro canal floodwalls. This testing was directed by an engineer from the New Orleans District (NOD).

Results were first published in 1988 (Jackson 1988). ILIT reports that the E99 test showed that

> "deep-seated movements in the levee foundation control the magnitude of the sheet pile deflection with the result that the height of water loading that can be sustained by a particular I-wall is controlled by the stability of foundation as determined by a slope stability analysis. It was concluded that conventionally determined deflections of the sheet piling were a poor criterion for design because movements were caused by deformations in the foundation and not the cantilever action of the sheet piles" (ILIT 2006, p. F-5).

Other results of the E99 tests showed that as the flood water level rose, the increased loading could cause a separation of the soil from the pile on the flood side that provided an avenue for entry of water. This water would introduce additional hydrostatic pressure on the wall, and cause failure at lower than previously predicted water elevations. Unfortunately, NOD engineers responsible for design of the 17th Street Canal floodwall and the others being designed at the same time, were not directed to consider this surcharge in calculating required sheet pile lengths and stable levee cross-sections. Both ILIT and IPET have concluded that a gap very similar to that observed during the E99 testing was involved in the initiation of failure of the I-wall on the 17th Street Canal and on the west side of the London Avenue Canal, at the northern breach.

Team Louisiana believes that while such failure mechanisms may have been involved, **the levees supporting the I-walls in all three Orleans Metro drainage canal breaches were simply too small in cross-section to resist the forces that were imposed on them,** whether because the underlying soils were too weak, or

because of reductions in strength associated with underseepage. It should be noted that all three I-wall breaches on the Orleans Metro drainage canals were through levees in very constricted rights-of-way that backed up on private property. This provided an apparently inviolable constraint on levee size for these canals, certainly if using I-wall designs. The same is not true of the levee on the Jefferson side of the 17th Street Canal or of levees with I-walls on the Orleans Avenue Canal that are larger in cross-section and abut a public road running along the landside toe.

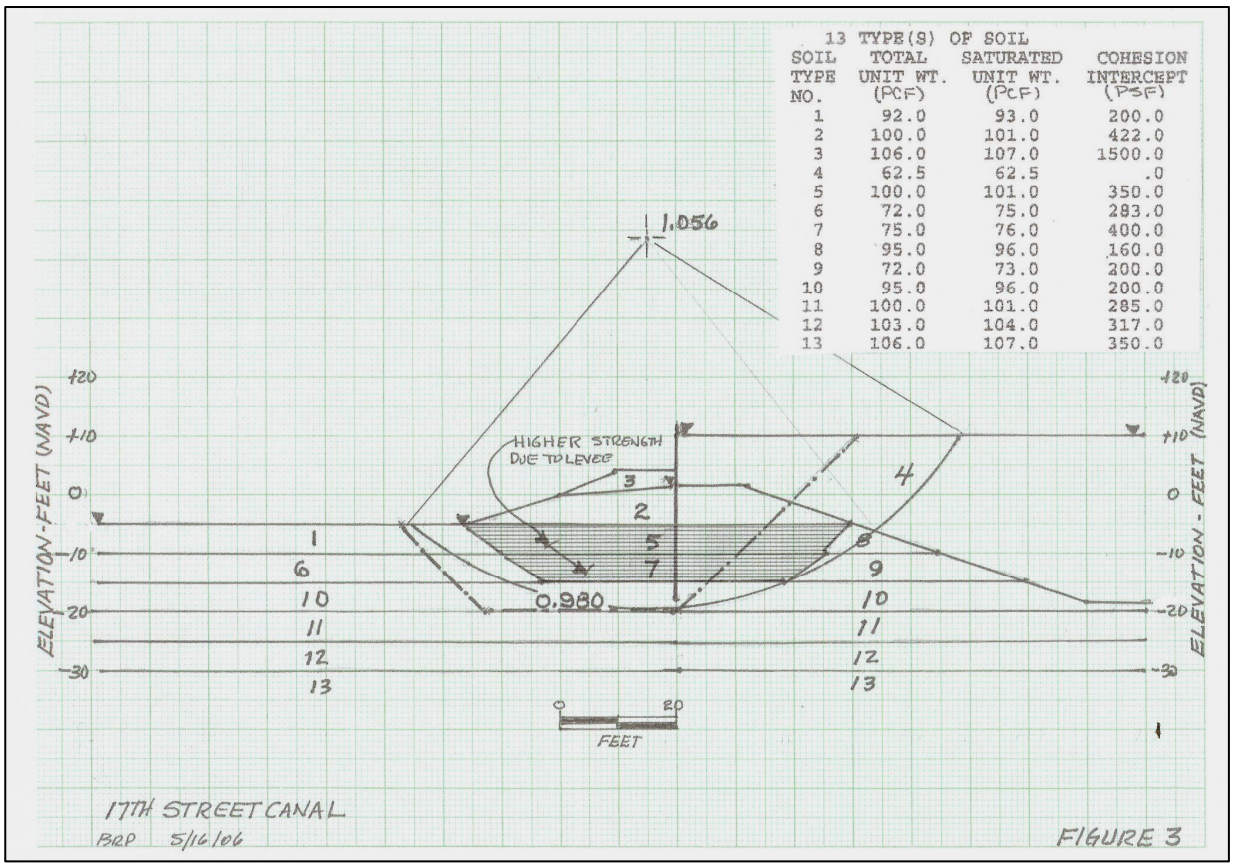


Figure 132. Stability analysis for 17th Street Canal breach area using generalized method of slices and using a canal water level of 10.8 ft, showing that the factor of safety goes to 0.98 (<1.0) indicating a zone of failure, at a depth of -20 ft. Cohesive strength and other soil and water parameters for various zones numbered on diagram are shown in table. Cohesive strength drops as low as 160 psf on canal side, but is 200 psf in the failure plane.

Both the London Avenue Canal and 17th Street Canal I-wall levees that breached had only a quarter of the cross-section of the Orleans Avenue Canal levee at the same relative location near the mouth, but had to resist the same surge and support I-walls of equal height (Figure 133). The Jefferson Parish side, or west

bank, levee of the 17th Street Canal is 3 times larger in cross-sectional area than the Orleans Parish levee that breached. Again, both levees had to resist the same forces (Figure 134). It appears that in some cases I-walls were put in levees that were too small and too weak to support the structure under the load for which it was designed. Elsewhere, as along the Orleans Avenue Canal, T-walls were used where needed and I-walls were embedded in levees with a larger cross-section.

One other consideration concerning the sheet pile at the 17th Street Canal is that the pile was driven through a cypress swamp. After the breach, large cypress tree stumps, as well as a log 3-4 feet in diameter, were exposed due to the topsoil having been removed. Team Louisiana had asked to be present when the remaining sheet pile was pulled (from the breach area) but were never notified. Our interest was whether in driving the sheet pile any had been damaged or torn due to encountering a stump or log. Any damage to the sheet pile sustained during driving would have affected its strength, and could have created a shorter subsurface pathway for seepage from the canal.

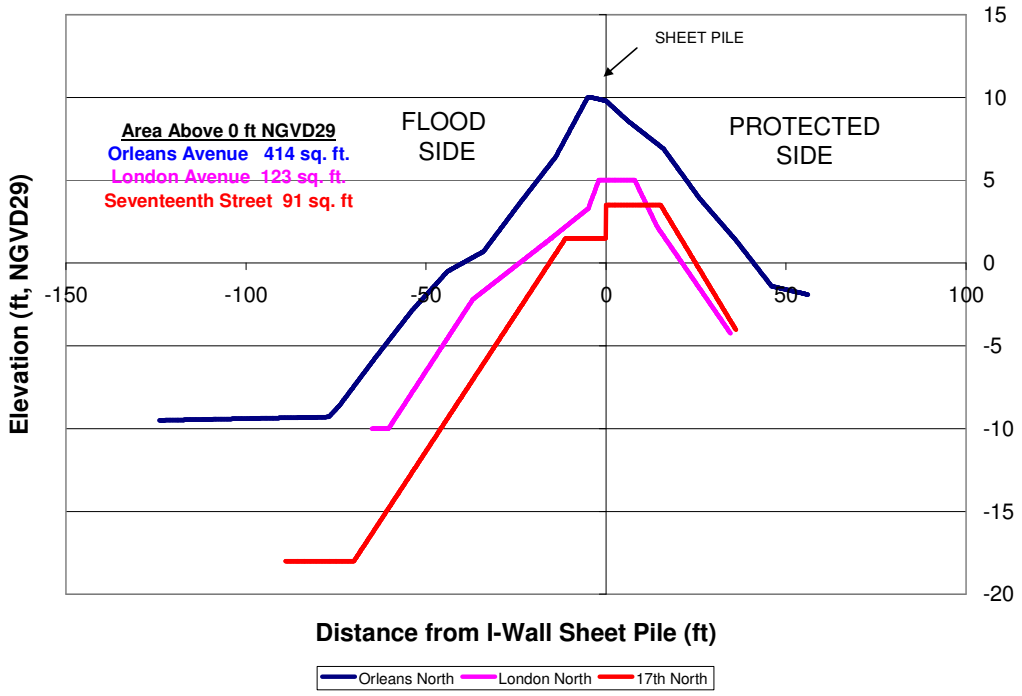


Figure 133. Cross-sections of I-wall levees on the 17th Street and London Avenue Canals that breached with the I-wall levee on the east bank of the Orleans Avenue Canal that survived at the same relative position on the canal.

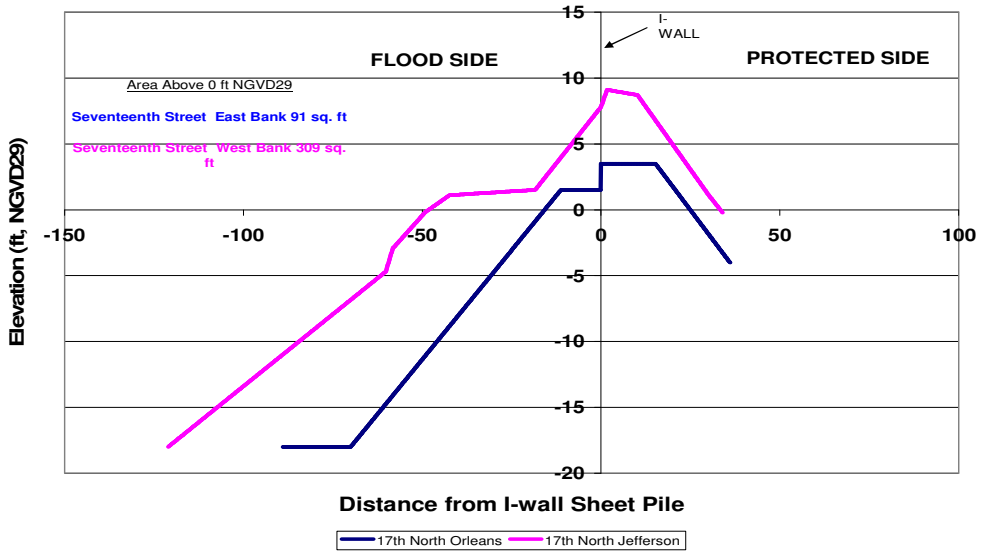


Figure 134a . Cross-sections of east I-Wall levee near mouth of the 17th Street Canal that failed with much larger I-Wall levee on the Jefferson Parish or west bank side. One approach to improving levee stability would have been to add a berm on the landside.

In trying to assess just what size the berm on the protected side of the 17th street canal should be to have insured against failure, the method of generalized slices was again used (Figure 134b). A earthen berm 54 feet wide with an average height of 4.5 feet produced a F.S of 1.4. While such a berm would effectively distribute the stresses to foundation soils, it would have required removal or demolition of numerous existing structures.

Armoring of earthen levees

The New Orleans District USACE did not follow standard engineering practice or Corps guidance when evaluating whether to protect (armor) earthen levees or sea dikes from erosion caused by waves in the funnel area east of the city. Such evaluations should have followed the 1954 TR-4 Shore Protection Planning and Design (Beach Erosion Board) or its successor, the Shore Protection Manual, first published in 1973.

Instead of the required analysis, Design Memoranda for the New Orleans East and Chalmette Levees substitute the following disclaimer.

> "Due to the short duration of hurricane flood stages and the resistant nature of clayey soils, no erosion protection is considered necessary on the levee slopes."

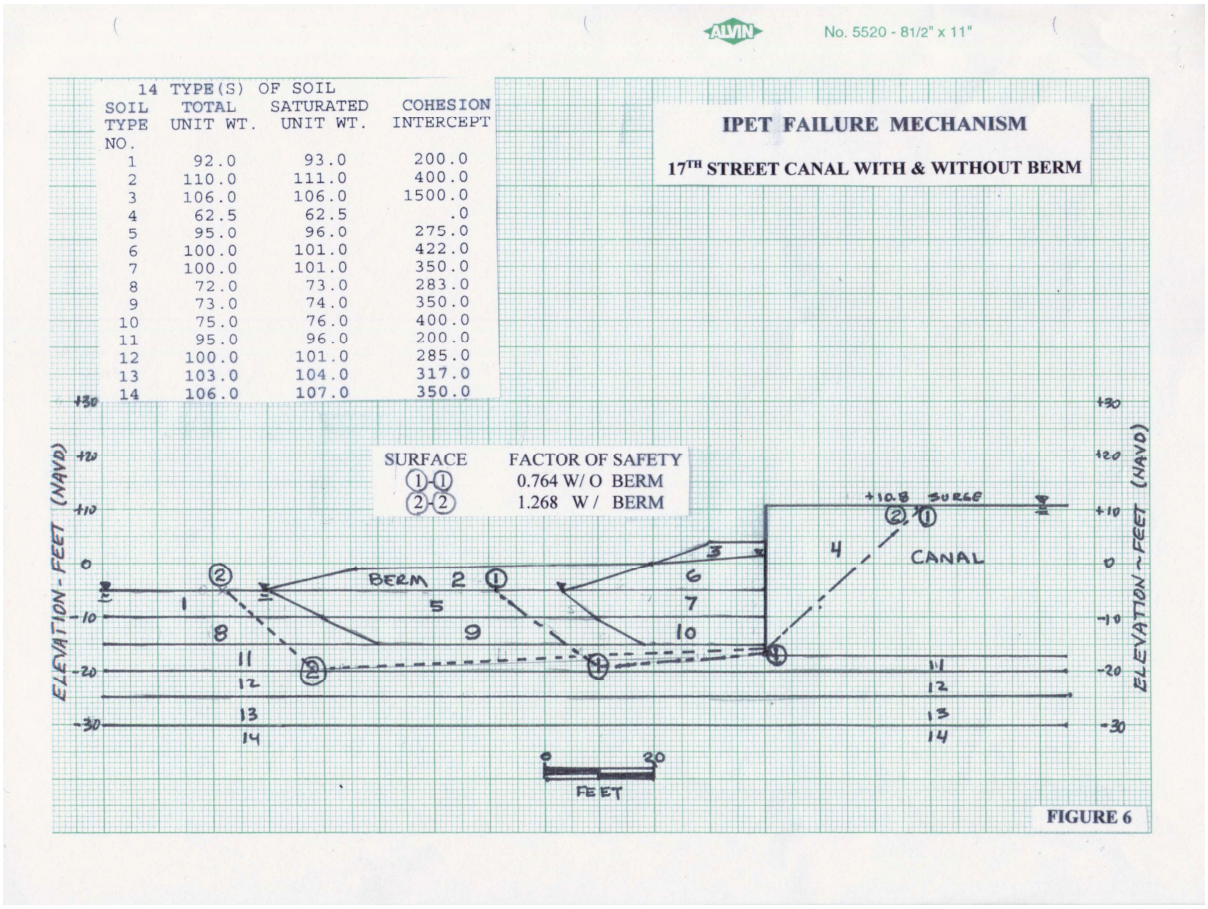


Figure 134b. Method of generalized slices used to determine levee size to ensure no failure at 17th Street Canal.

These levees were not designed to withstand general overtopping, as was amply demonstrated in Katrina, but they were expected to experience overtopping by the 14% of waves greater than the significant wave provided in the oceanographic analysis.

Based on visual observations in the field, post Katrina, many miles of the Chalmette and New Orleans East Levees were constructed of shell-rich sands derived from excavation of the adjacent GIWW and MRGO channels, rather than the clay soils specified for levees protecting urban areas (EM 1110-2-1913, 1978 ed.). This observation was also made by members of the ILIT team.

A simple concrete pad can stop overtopping erosion behind an I-wall. But no measures were taken within the GNO HPS to survive overtopping which, as we have noted, was equated to failure. Since Katrina, large and elaborate pads have been added behind all remaining I-walls in the GNO HPS expected to experience surge directly. This is perhaps the most cost-effective single measure undertaken

since Katrina to improve GNO HPS reliability quickly. T-walls experienced back side erosion also, but T-wall stability is relatively unaffected by the condition of the levee. The base of the inverted concrete 'T-' or 'L-walls' may in some cases extend through the back side plunge point providing some of the protection of a splash pad. Examples of armoring from a Dutch levees are shown in Figure 135a.

Investigation of Breaches on the West Bank of the IHNC

I-walls, T-walls and earthen levees provide protection for the Orleans Metro along the west bank of the IHNC or Industrial Canal. The IHNC was completed in 1923, dividing the 9th Ward neighborhood in half to create the Upper 9th Ward on the west side and the Lower 9th on the east side. It originally ran 5 miles from the Mississippi River to Lake Pontchartrain and was unconnected to other channels. At that time, water level in the IHNC was controlled by tides in Lake Pontchartrain, as is the case with the Orleans Metro drainage canals to the west. In 1944, however, the Gulf Intracoastal Waterway (GIWW) was rerouted to pass through the southern part of the canal, creating the first shallow channel through the wetlands to facilitate propagation of hurricane surge from Lake Borgne. Man had created a " Hurricane Alley" right into the heart of New Orleans. This opening was enlarged by a factor of more than 10 in the early 1960s by construction of the Mississippi River Gulf Outlet (MRGO), and this enlargement greatly enhanced the surge threat via the alley. This was first realized during Hurricane Betsy in 1965. We will discuss the role of the MRGO in surge enhancement in the next chapter. It is sufficient to state here that had surge been prevented from entering the IHNC from the MRGO/GIWW, the Industrial Canal would have seen 3 to 4 foot lower water levels during Katrina, and IHNC floodwalls and levees would have experienced little overtopping and a far lower likelihood of failure. Now that Congress has directed that the MRGO is to be closed, flood gates are being designed that will stop the surge, but, of course, the damage of Betsy and Katrina is already done.

All of the levee and floodwall reaches exposed to the Lake Borgne surge system, including those along the IHNC, experienced overtopping (Figure 135b). Water passing over the floodwall crest plunges 6 to 8 feet, developing velocities of 20 to 30 feet per second (fps) during the fall. These velocities are sufficient to erode a trench in almost any unhardened soil, but dissipate rapidly away from the plunge point. If flow continues for an extended period, the trench can get deep enough to compromise I-wall stability so that it begins to lean landward, allowing for a crack or gap to open up on the flood side. Once water can move down the floodside sheet pile face, it can increase pressure on the face of the

wall, and in any pervious materials like sand or peat can potentially migrate around sheet pile thus enhancing the failure.

Many IHNC floodwalls on the east and west banks that were overtopped performed well without splash pad protection. Compacted clay soils on the backside eroded slowly and trench development did not progress far before overtopping ceased as the surge dissipated. Elsewhere, the plunging water found more erodible sandy soils that allowed trench development. How far a particular overtopped floodwall progressed toward failure also depended on the local surge hydrograph, and on the extent of elevation deficiency, both discussed in earlier chapters. The time-elevation relationship on the flood side of a structure with a given crest or crown elevation determined the intensity and duration of over wall discharge available to work on removing the protected side of the floodwall levee (Figure 135b).

The Port of New Orleans (PONO) container facility on the west bank of the IHNC received the full impact of the surge jet entering from the east through the MRGO/GIWW ship channel (Figure 136). This area experienced the highest surge levels in the IHNC (Figure 137), up to 15.4 feet (NAVD88), and more than a foot higher than the 14.2 feet documented to the south at the IHNC lock gage. The surge diminished by at least 2 feet between the Port's container facility and Lake Pontchartrain.

Figure 135a. Examples of armoring of levees from the Netherlands. Left panel shows the use of rocks; right panel the use of a bitumized layer.

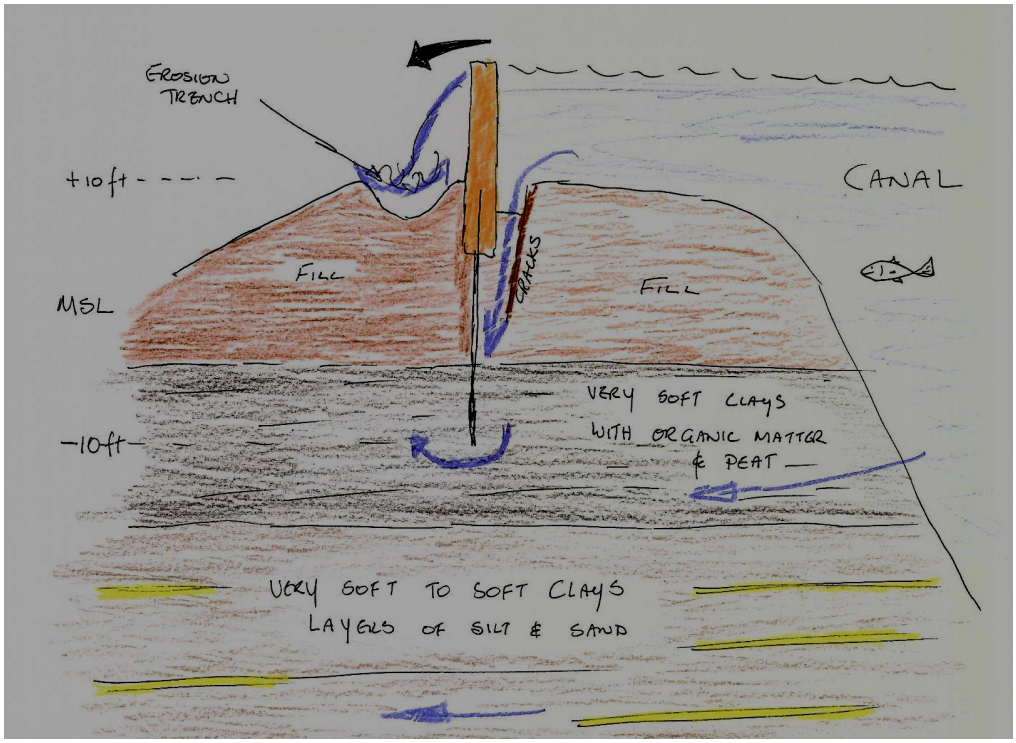


Figure 135b. Schematic of I-wall overtopping on the IHNC.

Figure 136. View northwest from St. Bernard toward west bank of the IHNC. Large, partially protected Port of New Orleans container facility is directly opposite entrance of MRGO ship channel, connecting the IHNC with the Lake Borgne funnel.

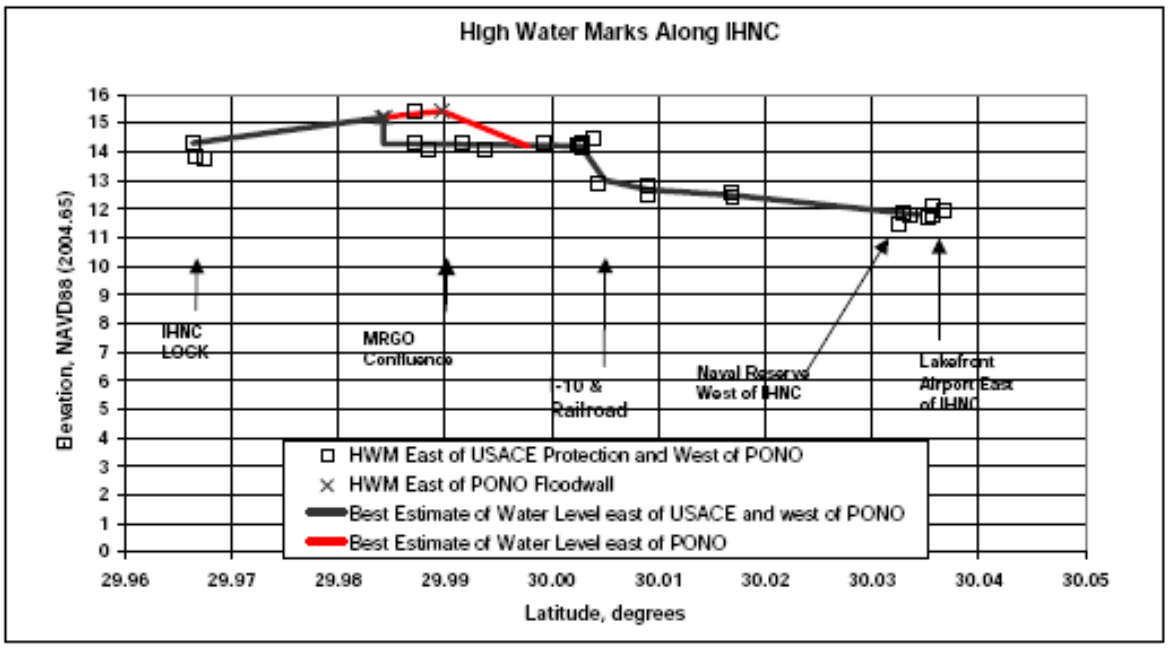


Figure 137. Trend in high water marks on IHNC west bank from IHNC lock on south to Lake Pontchartrain on north. Highest values in excess of 15 ft occur opposite MRGO entrance (IPET 2006b, IV-1-82).

It should be noted, however, that two lines of flood protection have been constructed in this area. The original USACE floodwalls and levee are set back from the canal bank and follow the south and west sides of the facility (Figure 138). The PONO has been constructing a bankside line of protection that is at least as high as the USACE HPS, and performed well in the storm, but it is incomplete at the northern end of the facility. The USACE I-walls and levees that failed at the PONO facility were exposed to surge that mainly came around the north end of the incomplete PONO floodwall (Figure 138).

If the IHNC floodwalls had been at the 14 to 15 ft (MSL) design level of protection, they would have experienced less than 1 ft of overtopping at peak surge, rather than the 1.5 to 4.0 ft that occurred. These structures were simply too low to provide the design level of protection (Figure 138).

A single I-wall reach was breached for about 200 feet north of the Benefit Street gate while T-walls in the same area sustained little damage. Scarring and cracking of the concrete I-wall caps in the breach north of the Benefit Street gate suggest that failure there could have been precipitated to some degree by impacts from containers picked up and swept against it by the surge (Figure

139). South of this gate, surviving segments of I-wall, however, showed typical signs of distress associated with overtopping including the flood side gapping

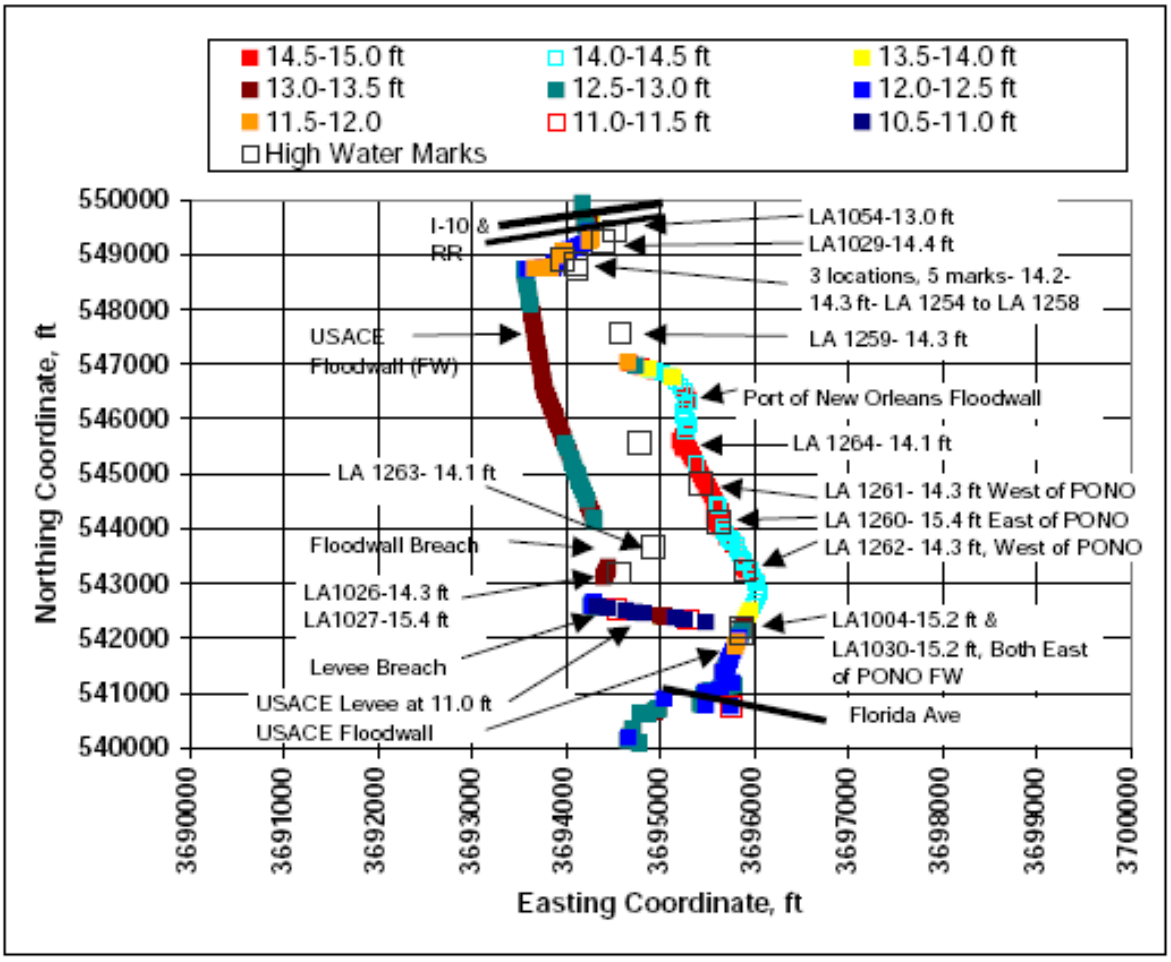


Figure 138. IPET surveys of levees and floodwall crown elevations surrounding the Port of New Orleans facilities on the west bank of the IHNC north of Florida Avenue railroad bridge. PONO bankside line of protection is incomplete at north end. Note 11 ft (NAVD88) elevations on levee along south side (IPET 2006b, IV-1-78).

that has been associated with most near-failures of GNO HPS I-walls (Figure 140).

T-walls and I-walls were intermixed on the west side of the IHNC to accommodate local needs for railroad and road access to port and industrial facilities built up to 5 to 10 feet on fill along the side of the canal. The 1968 GDM 2, Supplement 8, includes the following statement.

> "Because of the limited space available due to the proximity of roads, railroads, and existing industrial plant facilities, and the

Figure 139. View of I-wall failure north of Benefit Street gate at back of the Port of New Orleans container. Failure in this area may have been assisted by impacts from floating containers moving from right to left (IPET 2006b, V-14-64).

> necessity for providing protection against seepage and potential erosion, the protective works will consist of cantilever I-type floodwalls of steel sheet piling capped with a concrete wall where wall height is less than 10 feet, and T-type concrete floodwalls, with steel sheet pile cutoffs, supported by 12-inch by 12-inch square prestressed concrete bearing piles where the wall height is more than 10 feet" (GDM 2, Supplement 8, 1968).

The most serious flooding of Orleans Metro from the IHNC came not from failure of I-walls or T-walls, or even from general overtopping of the walls, but through weaker elements that were interspersed along the line of protection. The missing gate at the CSX railroad penetration just south of the I-10 bridge, where sandbags blew out, was discussed in an earlier chapter. Far more consequent was the failure of a 1,600 foot earthen levee that runs along the south side of the France Street container terminal as part of the USACE HPS (Figure 141). IPET (2006b) surveys show that this levee had a crown elevation of 11 ft (NAVD88), about 4.5 ft below the 15 ft (LMSL) design level of protection and 4 ft

below the Katrina surge in this area (Figure 138). This levee was a particularly low part of the IHNC west bank HPS and began to be overtopped at about 0530.

Figure 140. Floodside gap on I-wall that has rotated landward in an incipient failure mode. View south from Benefit Street gate toward France Road ramp. Earth levee that failed is in the background to the left of the road (from IPET 2006b, V-13-17).

The container terminal is a very complicated area to protect (Figure 142). The USACE levee along the south side runs west and under the built up roadbed of the France Street Ramp to link up with an I-wall. A major breach in the levee just east of the ramp was followed by erosion and removal of the porous sand and shell fill of the road embankment on the outside of the levee (Figure 143a). Water flowed along the outside of the levee and washed out the ramp developing a large scour hole. Erosion of the granular ramp materials created a white depositional fan that was clearly visible from the air immediately after the storm (Figure 141).

Figure 141. View southwest of breach in earth levee running from the IHNC to the France Road ramp along south side of the PONO Terminal facility. Depositional fan contains gravel and shell eroded out of the roadbed built up to cross the levee. Enlargement in lower panel shows flow through ramp was concentrated along outside of earth levee but that ramp formed a transition to an I-wall.

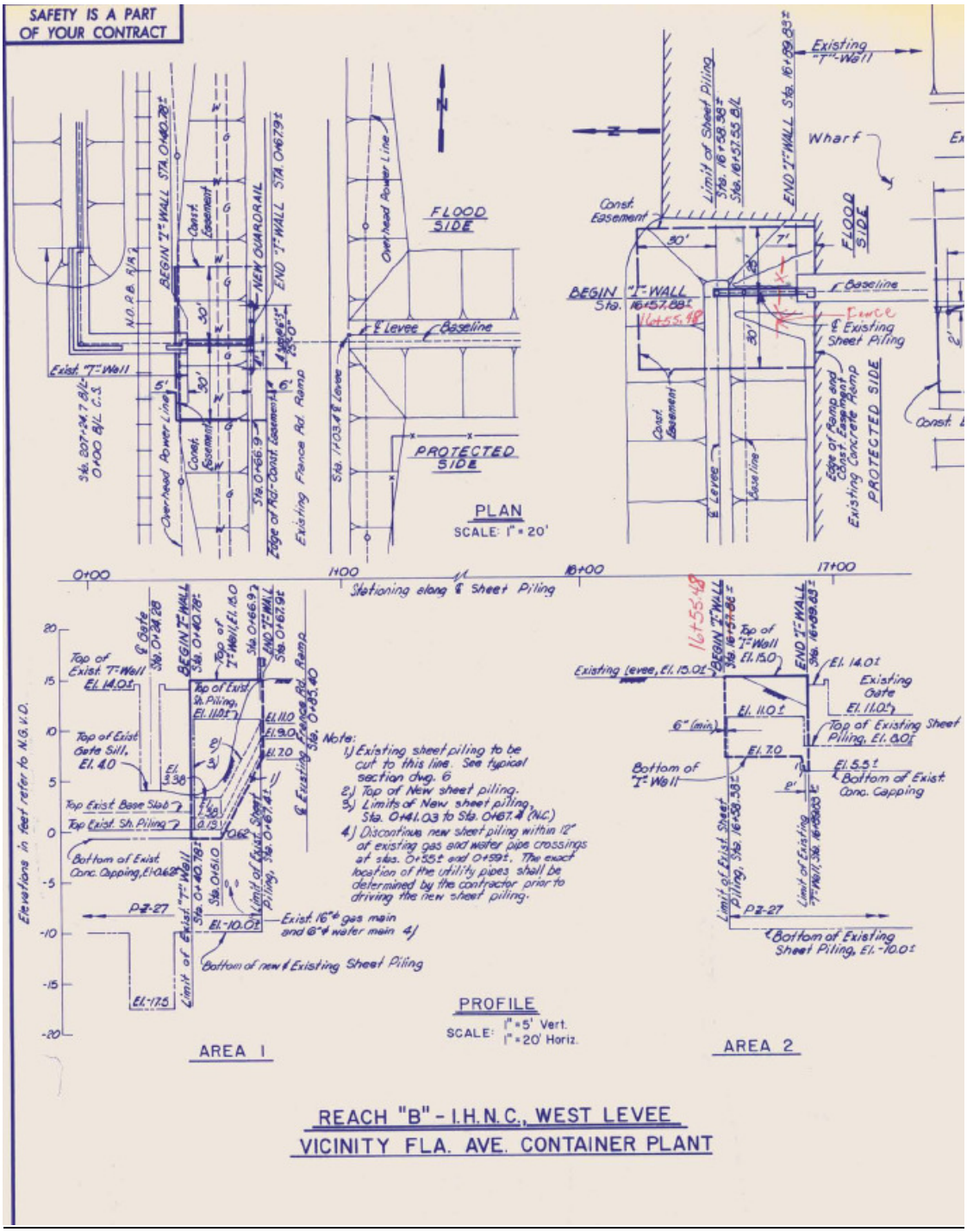


Figure 142. 1983 As-built drawing of east and west ends of levee along south side of France Street container terminal. 'Area 1' detail on left shows the tie-in with France Road Ramp, with I-walls and with the railroad.

Figure 143a. Erosion of sand and shell fill of France Street Ramp roadbed at junction with I-wall to right on south side of PONO container terminal north of Florida Avenue.

The long USACE levee that overtopped and breached on the south side of the container facility contributed more water than any other IHNC breach to the flooding of the Desire/Upper 9th Ward neighborhoods on the south side of the Metairie/Gentilly ridge. The breaching here happened before Katrina made landfall in Louisiana, about two hours before general overtopping and also before the floodwall failure farther north. The levee was low and poorly integrated into the adjacent floodwall through an erodible roadbed that was not designed to serve as a flood protection element. After the storm, the remains of the levee were removed and it was replaced with a T-wall structure.

The incomplete PONO levee along the canal bank probably lowered the surge experienced at the breach location by about a foot, but could have prevented it entirely if the bankside system had been fully deployed. The PONO levee is a step toward the redundancy that is so critically needed in this system, but if redundancy is a goal that is seen as an expensive luxury, it will continue to be difficult to see such projects through to completion.

Lower 9th Ward Floodwall Failures

Two breaches occurred in I-walls on the east side of the IHNC into the Lower 9th Ward (Figure 144). T-walls were not used on the east bank between the Florida Avenue and North Claiborne Avenue bridges, and do not appear to have been proposed for this floodwall reach. If they were not proposed, this was presumably because sufficient room was available to construct a levee large enough to ensure that wall exposure would be less than 10 feet. The north breach occurred just south of the blue Florida Avenue railroad bridge, and was less than 200 feet wide (Stations 54 to 56), while the south breach was almost 1,000 feet across (Stations 22 to 31). These I-walls were built in 1969, and were among the first projects constructed under the 1965 GNO HPS authorization.

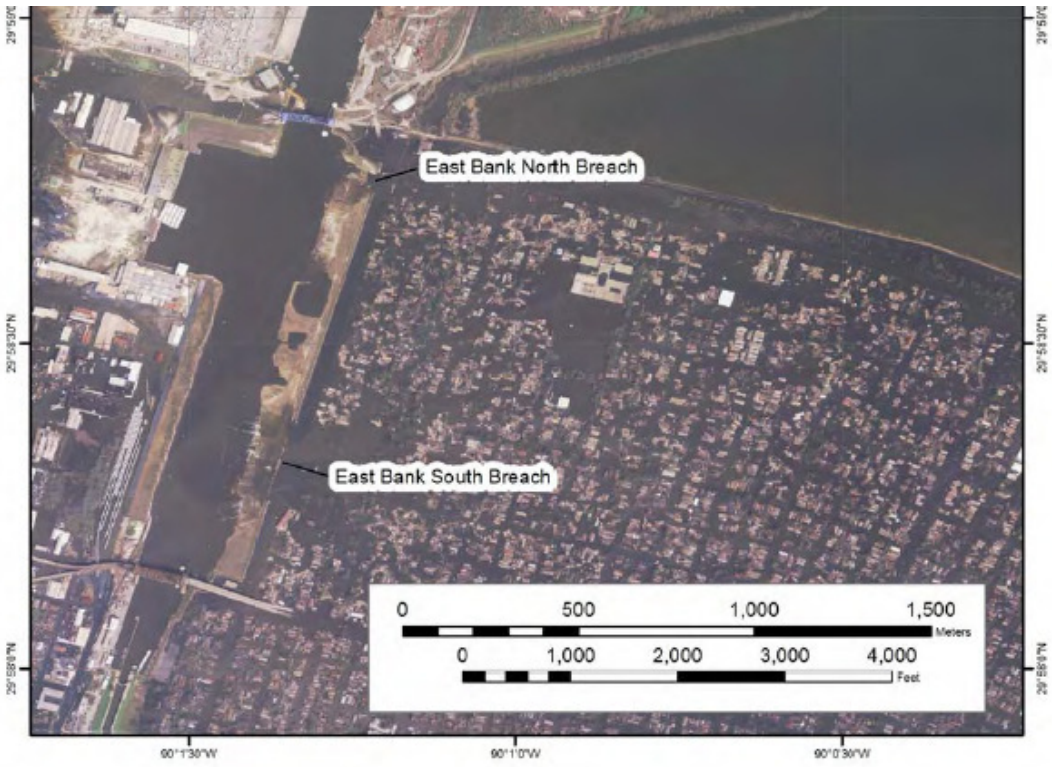


Figure 144. Aerial view of IHNC showing two breaches into Lower 9th Ward on the east side between Florida Avenue (top) and North Claiborne Avenue (bottom) bridges. Lock is visible at south end of IHNC (IPET 2006b, V-11-18).

The banks of the IHNC on the east side outside the I-wall alignment are higher than the land behind the levee, having been built-up with materials dredged from the Canal. This sill extends 100 to 200 feet west of the I-wall alignment at about +2.5 ft (NAVD88). A single I-wall template was used for the entire reach covered by both breaches (Figure 145).

An existing embankment was to be enlarged and piles driven to a tip penetration depth of -8 ft. Concrete I-wall monoliths were constructed to achieve a design level of protection (in 1969) of 15 ft MSL. Since the 1965 adjustment for zero NGVD29 rather than local sea level was used to control construction (Figure 146), everything was built 1.7 feet too low, even though elevations on the as-built drawings for this project are noted relative to MSL. These datums issues are discussed in more detail in this report.

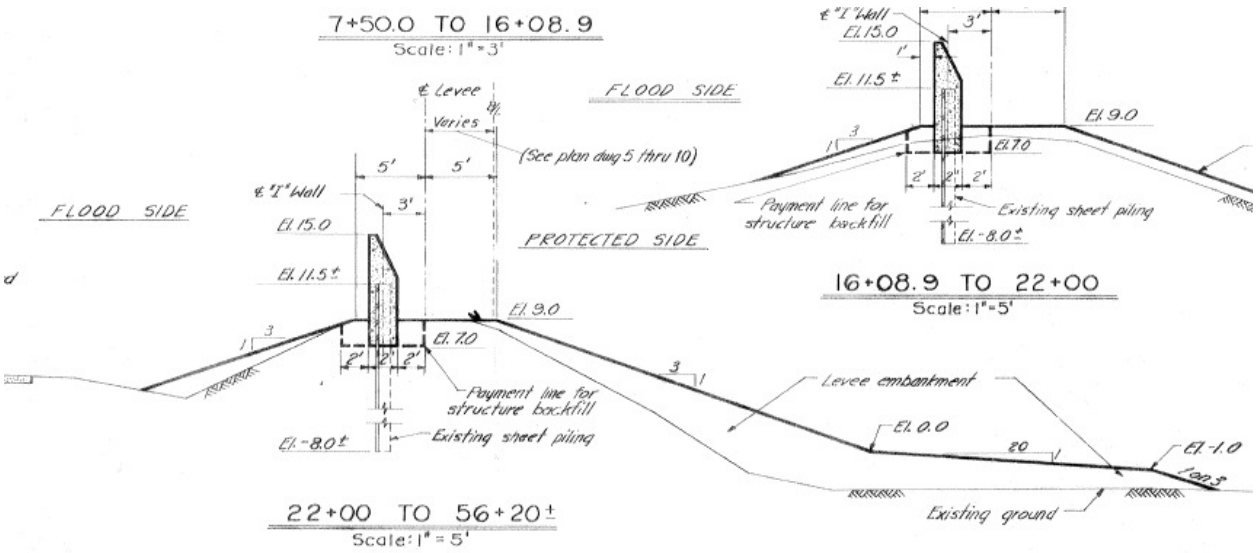


Figure 145. As-built drawings for a typical I-wall installation on the east bank of the IHNC adjacent to the Lower 9th Ward. The reach referenced in the lower drawing covers both north and south breaches.

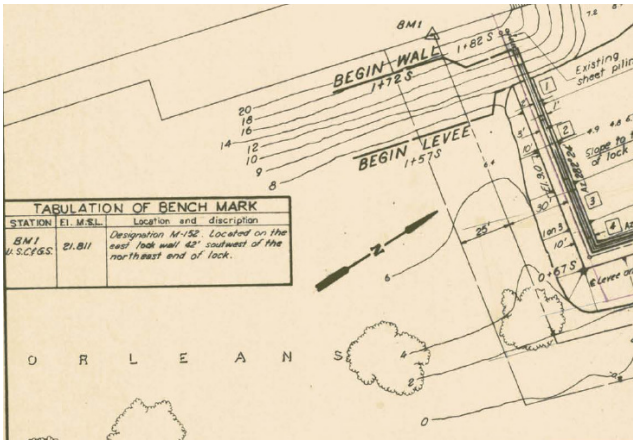


Figure 146. Portion of 1969 as-built drawing showing the elevation of the controlling benchmark as 21.811 ft MSL. This elevation was actually the 1965 NGVD29 adjustment for this benchmark. MSL at this time was 1.65 ft higher.

The geology under the east bank IHNC I-wall is similar under both breach locations, with clays extending from the surface to about -60 feet, and a relatively thin layer of sands below this point, with more clay beneath, to the top of the Pleistocene at -80 feet (Figure 147). The I-wall levees sit on a thin veneer of fill from the construction of the canal, but a marsh or swamp sequence nearly 20 feet

thick that appears to be composed of at least two cycles with different geotechnical properties provides the primary foundation. Only the top few feet of the marsh are penetrated by the sheet piling. Again, as with most I- wall designs, it is not necessary to look very deep to discover a problem, if there is one.

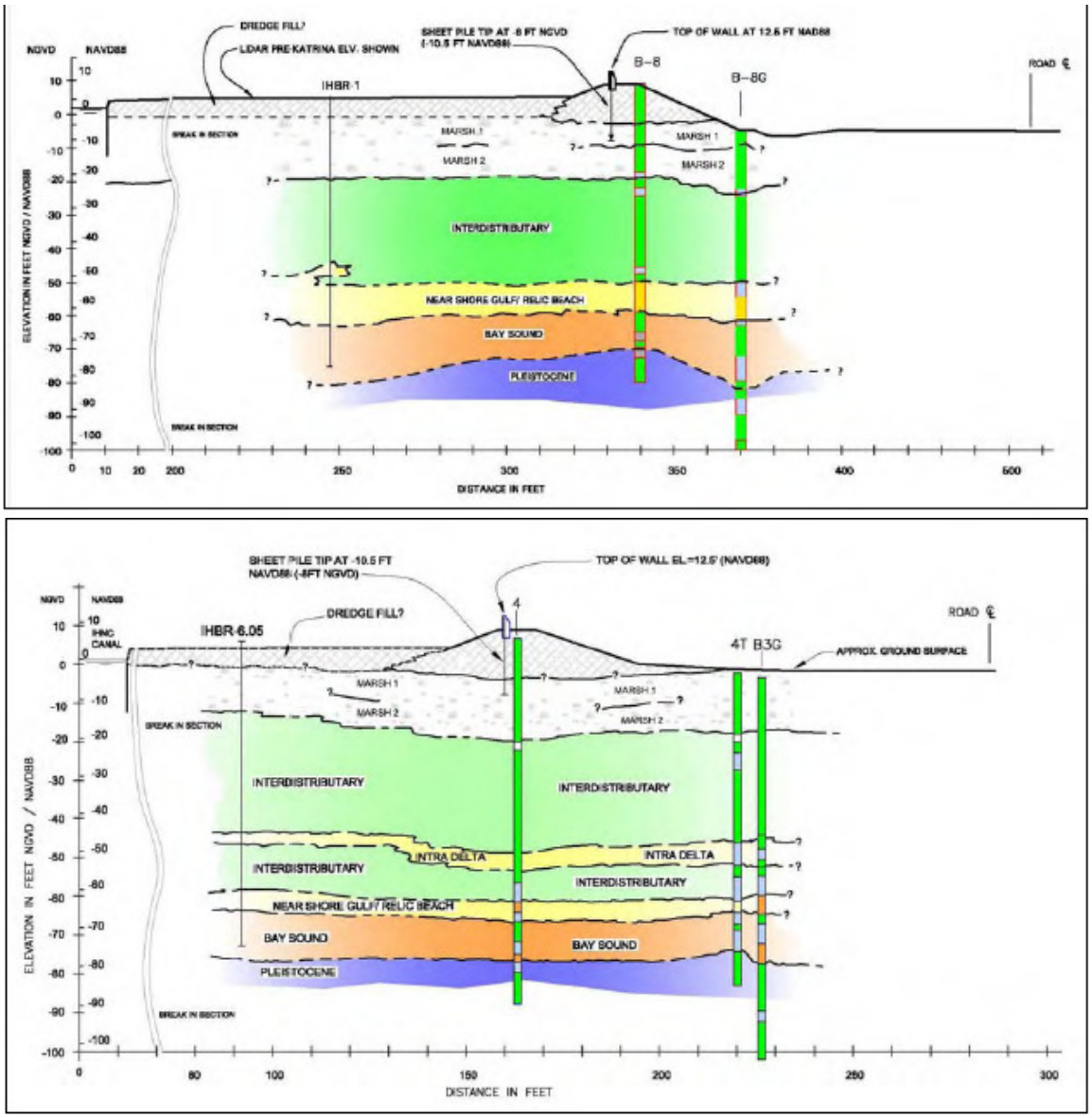


Figure 147. Stratigraphic cross-section under the north IHNC east bank breach (top), and (bottom) under the south breach (IPET 2006b, V-11-31). Protected side is to right.

While the land is built up with fill from the canal on the flood side, this fill does not extend inland past the levee alignment, and the protected land is

considerably lower. Land elevation on the protected side trends down going away (north) from the Mississippi River natural levee into drained wetlands, as was true along the Orleans Metro drainage canals (Figure 148).

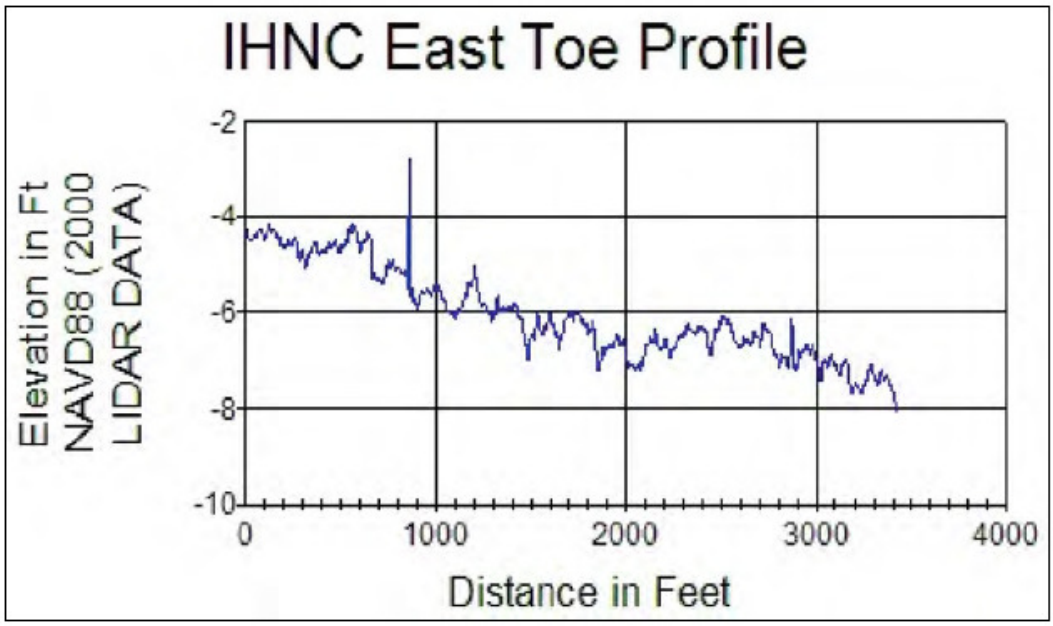


Figure 148. Land elevation from 2000 LIDAR data for protected side levee toe from south end of south breach on left to north end of north breach on right. The toe elevation in south breach area averages -4.5 ft (NAVD88), while that behind north breach is -7 ft (IPET 2006b, V-11-28).

The approximately 3 foot difference in land elevation at the protected toe can be seen in cross-sections of the levee derived from as-built drawings (Figure 149). A single design template was used to construct the I-wall levee for the 2,400 ft reach that includes both breaches (Figure 145). The 1969 levee was constructed over an earlier, smaller embankment. The I-wall levee constructed was about equal in cross-section to that built on the Jefferson side of the 17th Street Canal, approximately 300 square feet above 0 ft (NGVD29) (See Figure 149).

IPET has determined that the relatively small difference in elevation at the protected toe in the north breach area was sufficient to compromise stability there before the surge reached the top of the floodwall (Figure 150). In a slope stability analysis similar to the Method of Planes method used by the original designers, a factor of safety of near unity (failure condition) was found at a surge elevation of 11.2 ft (NAVD88), about 4 ft below the maximum surge experienced in this area, and more than 1 foot below the elevation of the adjacent I-wall (12.5 ft NAVD88).

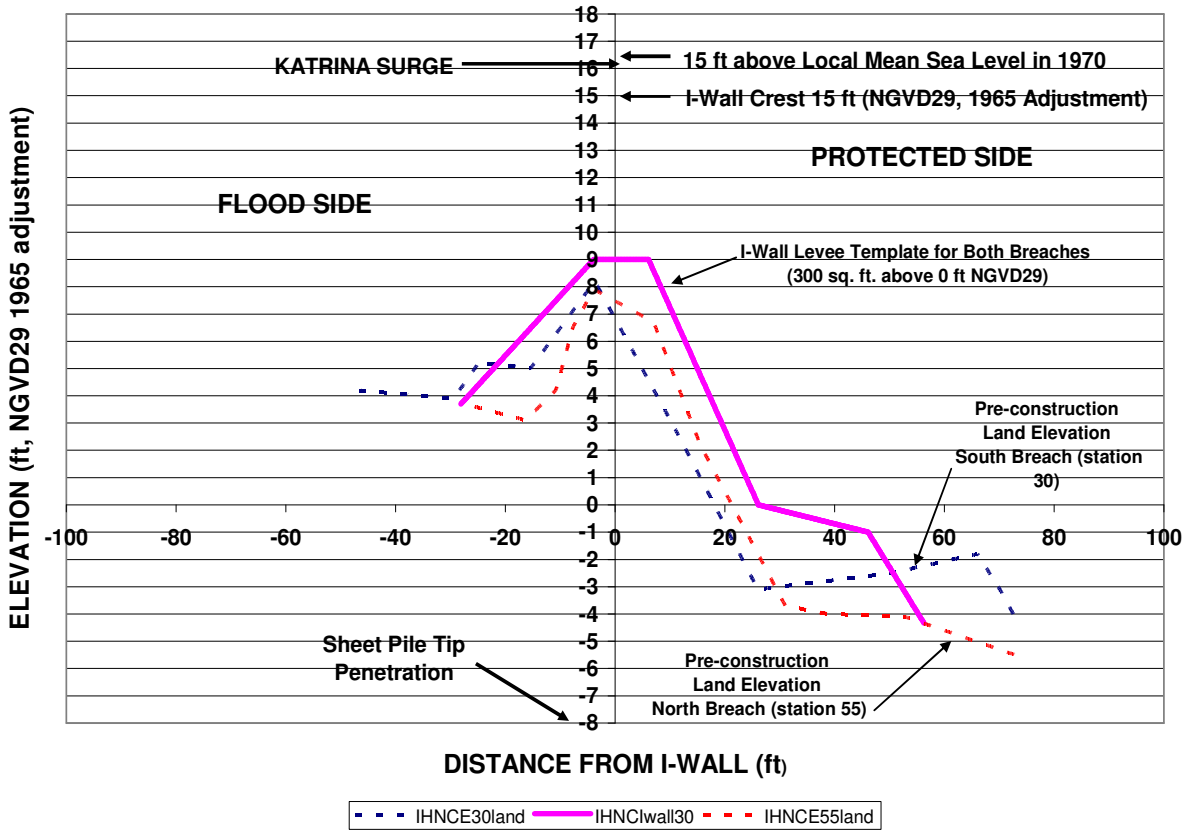
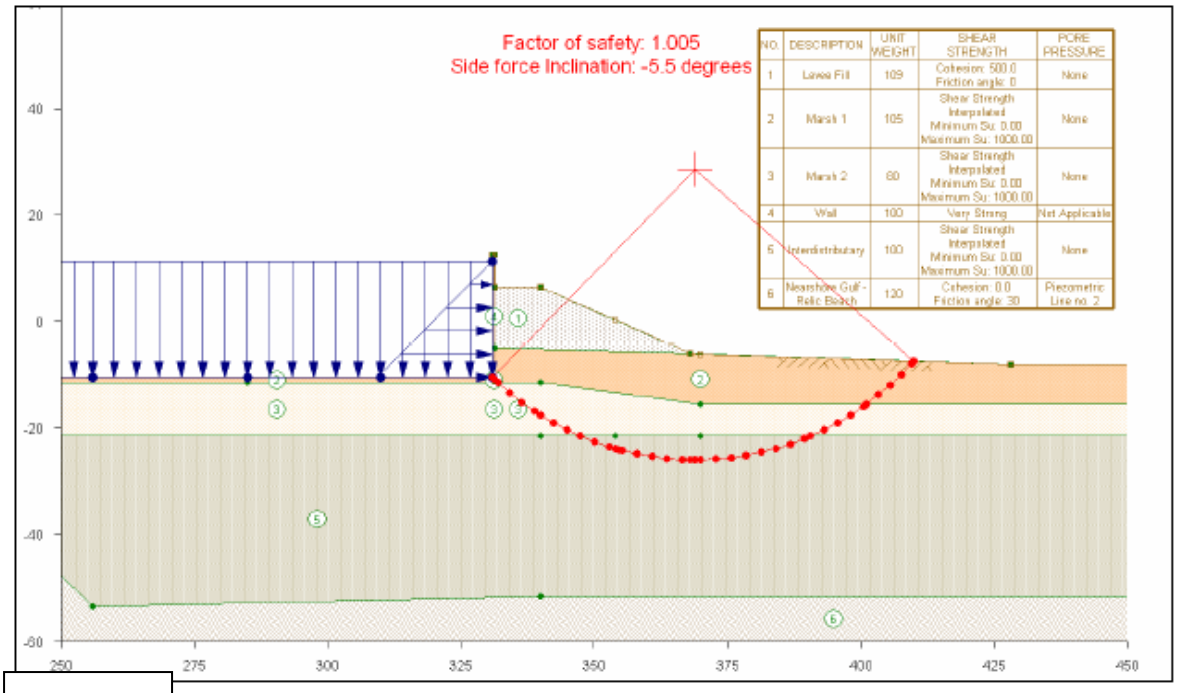


Figure 149. Overlay of the as-built I-wall levee template for stations 22 to 56 on pre-construction surveys at station 30 (south breach) and station 55 (north breach). Note 3 foot difference in land elevation at protected side toe (+60 ft). I-wall levee is about 300 sq. ft. in cross-section above 0 ft (NGVD29). As-built and design level of protection shown at top of I-wall, with Katrina surge level.

A condition of failure at this canal elevation was that a crack developed on the flood side of the sheet pile allowing the hydrostatic force to be exerted to the tip of the sheet pile. (Figure 135). The failure occurs through the more consolidated soils under the levee and extends just below the marsh unit (Figure 150). In contrast, at the south breach site, IPET determined that the wall should have remained stable with a factor of safety greater than 1 with water up to the full elevation of the top of the wall (12.5 ft NAVD88) even with a crack to the tip of the sheet pile (Figure 151).

We saw abundant evidence of flood side cracking and sinkhole formation in the surviving I-wall segment between the north and south breaches (Figure 152). In some cases, it appeared that the wall had rotated in place, while in others it appeared that some landward translation had also occurred (Figure 153).



HNC – East Bank (North Breach), Case 4, Canal Water Level = 11.2 ft (NAVD 88), with Crack

Figure 150. IPET stability analysis for IHNC east bank north breach. Wall was stable unless crack formed on flood side (IPET 2006b, V-11-34).

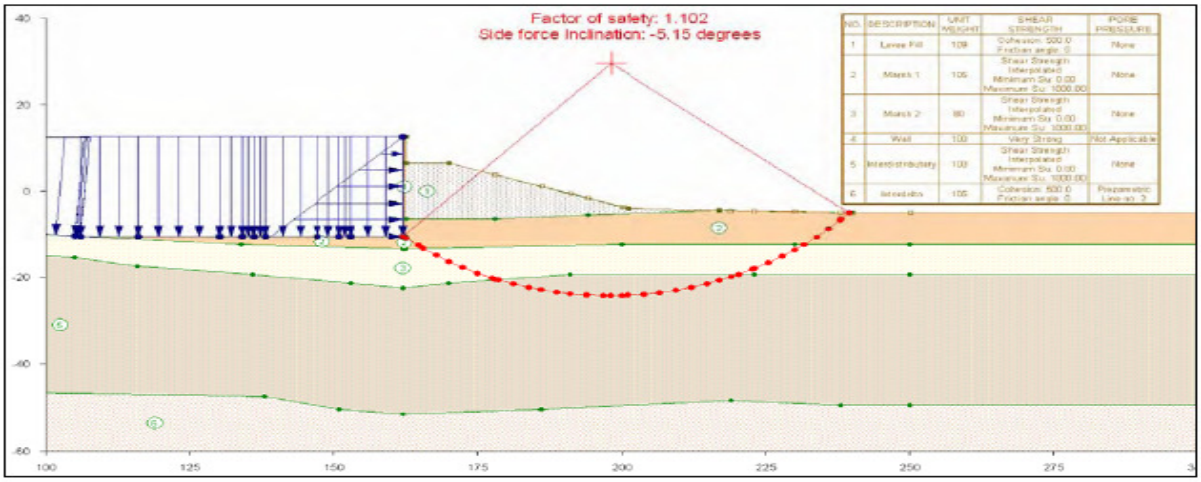


Figure 11-29. IHNC – East Bank (South Breach), Case 4, Canal Water Level = Top of Wall - 12.5 ft (NAVD 88), with Crack

Figure 151. IPET stability analysis for IHNC east bank south breach. Wall is stable (FS>1) even with crack formed on flood side (IPET 2006b, V-11-38). It should be noted that the circle analysis used by IPET is less conservative that the method of Planes used by NOD in method of generalized slices used by Team Louisiana (see Figure 132 for example of 17th street stability analyses results).

Note that this analysis has the geometry wrong as IPET does not include the drainage ditch at the foot of the levee. Additionally, they should have used the generalized method of planes rather than a circle.

Figure 152. Crack with sinkholes beneath un-deformed sod on flood side of IHNC east floodwall near north breach.

Figure 153. Deformation of surviving I-wall between the north and south breaches on east bank of the IHNC. Wall has both rotated and translated with percolation cracks in on the flood side and erosion trench on protected side.

We have seen the GDM and as-built drawings for the I-walls that failed (Figure 154). The slope stability analysis from the GDM discussed by IPET indicates that the mechanism of flood side crack formation was not anticipated in the late 1960s. On the other hand, the cantilever sheet pile stability analysis in the same GDM might offer some insights. We have not seen discussion yet of the possibility that a floodside crack might start with sheet pile deflection, rather than occur solely as a slope stability failure. Such cracks had been noticed by engineers in the offshore industry as reported at an Offshore Technology Conference in 1969 by Matlock and Reese.

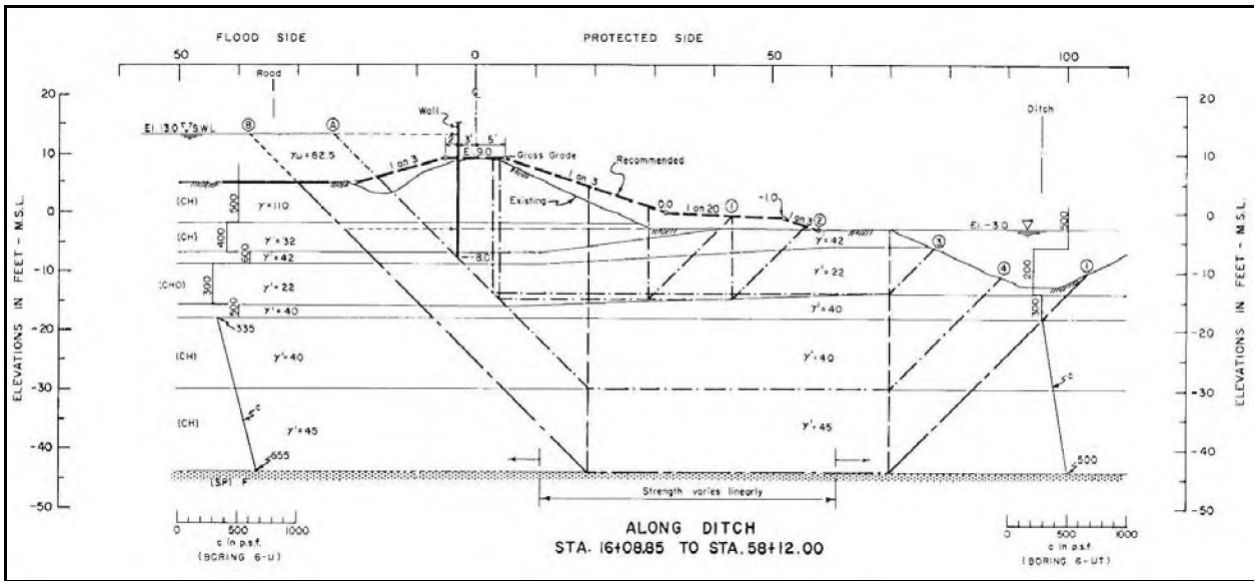


Figure 154. Slope stability analysis apparently from IHNC east bank GDM showing soil strength under the levee crown and landside toe (IPET 2006b, V-11-28).

Most investigators believe that the south breach formed mostly as a result of backside levee removal through erosion caused by plunging flow over the top of the I-wall. However, seepage beneath the sheet pile was also very important. The scour trench behind surviving I-walls on either side of this breach enlarges into the breach, and the remnant of a large trench was still visible on the land side of the original alignment in the breach itself (Figure 155). It is more difficult to explain why this particular 1,000 foot reach gave way, while adjacent segments did not. The surge elevation actually increased from south to north toward the MRGO confluence, so it is unlikely that this was the cause.

It is more likely that the I-wall crest elevation was lower within the breached reach, perhaps as a result of local settlement. IPET uses a -10 foot (NAVD88 2004.65) tip elevation for the stability analysis of this breach, which is about 0.5 foot below the -9.5 foot value that would result from converting the -8 foot as-

built elevation (NGVD29, 1965 adjustment) to the current datum (Figure 156). A 0.5 foot increase in sheet pile penetration would translate to a 0.5 foot decrease in top of wall elevation. If all or some of the wall in the breached reach had settled half a foot lower than surviving adjacent I-walls found to have a 12.5 foot crest elevation, this could have made the difference, resulting in onset of overtopping a half hour earlier (Figure 157).

Figure 155. Remains of scour trench excavated behind I-wall by plunging flow over the I-wall at the south end of the south breach on the east bank of the IHNC. The exhumed concrete-capped sheetpile from the I-wall is visible lying to left.

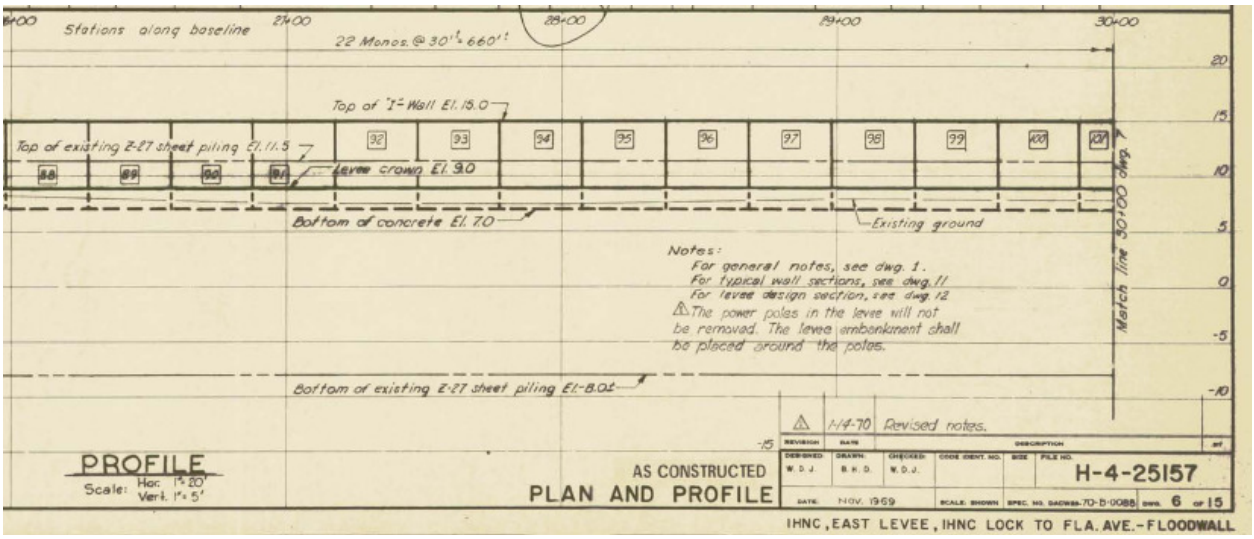


Figure 156. Portion of as-built drawing for I-wall in south breach area showing sheet pile penetration to -8 feet (NGVD29, 1965 adjustment). This corresponds to a tip elevation of -9.5 ft referenced to NAVD88 (2004.65).

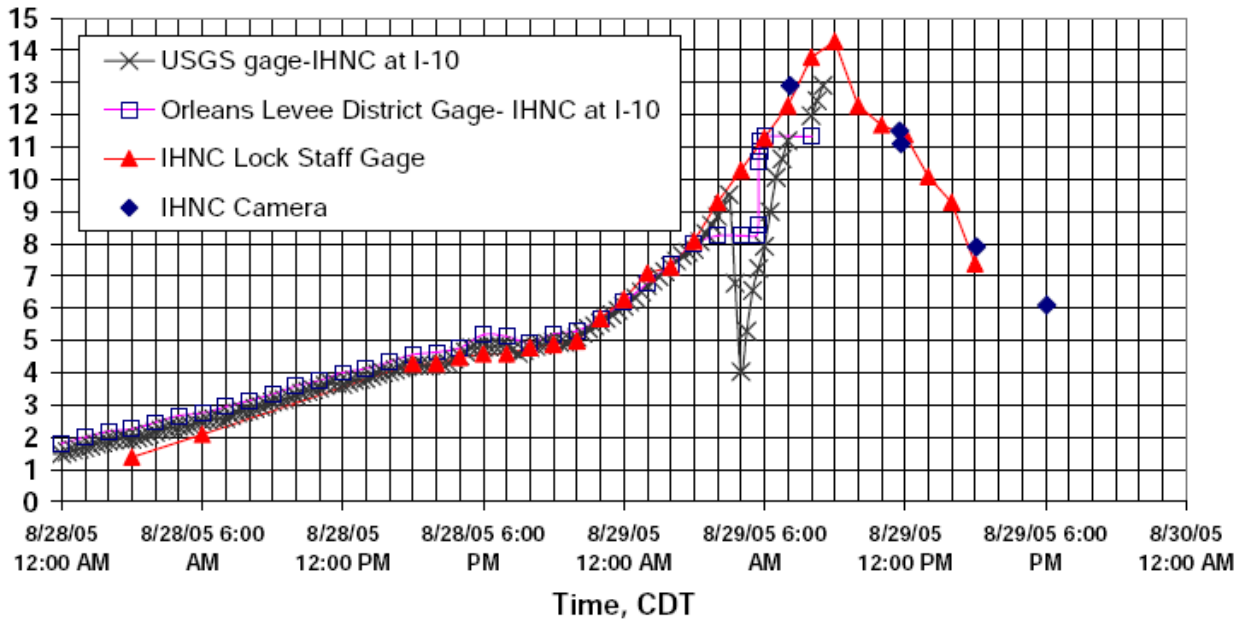


Figure 157. Gage records from IPET (2006b, Vol. IV-33). Red line indicates record from the IHNC lock south of the IHNC east bank breach sites.

A second possibility is that the soils on the back side of the breached segment were more erodible than those found in adjacent levees. Unfortunately, very little information is available regarding the materials used to construct the levees. IPET (2006b, V-11-15) reports that the levee fill at the north breach site was clay, but at the south breach location was a mixture of silty sand and clay. It would take just a short section with a particularly low crest elevation or unusually

erodible material at the levee crown on the protected side to initiate failure at one spot that could expand north and south.

Because these breaches occurred *before* the peak of the surge, the Lower 9th Ward neighborhood four feet *below* sea level was drowned by water as high as 14 feet *above* sea level. Many homes were washed off their piles, while hundreds flooded to the roofline and thousands of people were trapped. The wave of water coming through these breaches explains the total devastation of the homes within five blocks of the breaches—devastation very similar to what was occurring on the Mississippi shoreline at about the same time. Hundreds drowned in the space of about 90 minutes. An empty barge floated into the south breach knocking the top 9 inches off the concrete wall that had already failed.

Had the levees been equipped with back side splash pads on the land side, it is possible that the south breach might not have occurred, and flooding of the Lower 9th Ward would have been less rapid and dangerous. The breached areas and the intervening I-walls that survived have now been replaced by a new T-wall, with pilings driven to the Pleistocene at -80 feet. The floodwall crest elevation has been built to the +15 LMSL elevation of the original design but still transitions to adjacent walls 2.0 feet lower. While protection is restored to the area by this new wall, the protection provided still does not meet the criteria of the 100 -year project storm.

Other floodwalls on the east and west banks of the IHNC overtopped into New Orleans East and into Orleans Metro closer to the Lake. Nowhere else, however, did they fail completely. We discuss the impacts of still higher surge elevations during Katrina on earth levees and floodwalls surrounding the MRGO and Lake Borgne funnel in the next chapter.

CHAPTER SEVEN

WAS THE HURRICANE PROTECTION SYSTEM EAST OF THE IHNC COMPROMISED BY POOR INTEGRATION OF NAVIGATION CHANNEL AND LEVEE DESIGNS?

Question 4. Did the MRGO, a free-flowing, deep-draft navigation canal that pierces the HPS on its eastern side, compromise system performance?

Answer 4. Yes. The MRGO and GIWW channels provide efficient conduits to funnel surge into the heart of New Orleans. As a result, surge elevations peaked in Lake Borgne and the IHNC at nearly the same time, and at higher levels relative to levee and floodwall crowns than would have been true if the MRGO did not exist and had not caused so much wetland loss. The effect of these federally constructed and operated channels on surge and waves has consistently been underestimated by the USACE from before Hurricane Betsy right through to the recent IPET report, as has the effect of accelerated wetland loss in the funnel area. One consequence of this institutional "blind spot" was that a hurricane barrier of the type proposed in the original pre-1980s HPS for the other two main passes into Lake Pontchartrain was never even considered for the MRGO.

The ILIT and IPET have indicated that the original "barrier" approach was a better design than the "high-level," levees-only HPS ultimately adopted twenty years after authorization. But our work indicates that disastrous flooding during Katrina into two of the three GNO basins (New Orleans East and St. Bernard), those adjacent to the Lake Borgne funnel and the IHNC, would have been exacerbated by the barrier proposed at the Lake Pontchartrain terminus of the IHNC (Seabrook). On the other hand, the Lake Pontchartrain surge along the south shore might have been reduced by up to 3 ft by the barriers that were proposed for the other Lake Pontchartrain passes. This might have been enough to prevent one or more of the failures of the defective Orleans Metro drainage canal floodwalls built in the 1990s, and this would likely have reduced the severity of flooding in Orleans Metro. Certainly, the barrier plan would also have reduced surge along the north shore of the Lake by 4 to 5 feet. Such trade-offs were never rigorously assessed when the decision was made to change the HPS design in such a radical way at a time when surge modeling techniques using SLOSH were available. Again, the level of protection was reduced without informing the population at risk.

All existing federal navigation projects in coastal areas should be evaluated by independent investigators using modern modeling techniques to determine the effect on existing storm flood protection. This can form the basis for FEMA recertification or a remedial retrofit plan. The MRGO and other channels in the funnel must immediately be fitted with appropriate closure features to ensure that it does not serve again as a conduit for surge and waves to GNO

Barriers and Channels East of the IHNC

The landscape of the GNO east of the IHNC is a maze of contradictions. Levees built to stop storm surge sit incongruously next to ship channels that speed it through a 'funnel' into the heart of the City (Figure 158). Navigation channels like the GIWW and MRGO facilitate trade, but are liabilities when storms threaten. Levees are expensive nuisances that interfere with water access and cause environmental damage but are indispensable during storms. Both are federally funded projects, designed and built by the USACE. The MRGO and the GNO HPS were planned by the USACE at the same time, and were projected to have similar construction costs, about $90 million each, but the effect of one project on the other was not considered until after MRGO construction began in 1958.

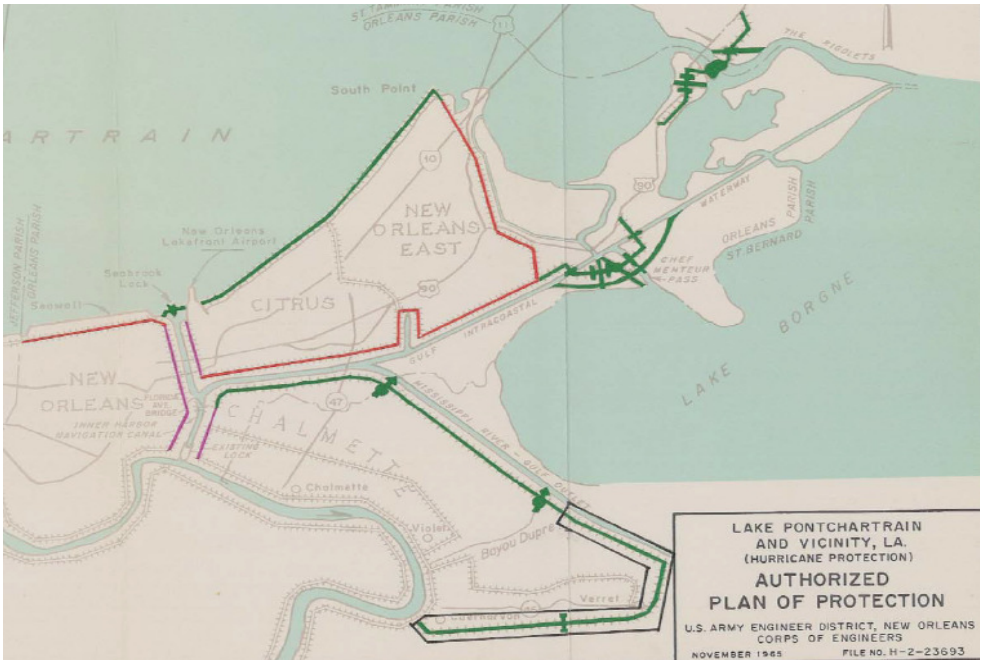


Figure 158. Creation of the 'funnel,' as shown in 1966 GDM 3, Supp. 1, for the Chalmette Levee Extension. GNO HPS authorized in 1965 was quite different from what has actually been built. It included surge barrier gates at the two passes from Lake Borgne into Lake Pontchartrain (Rigolets and Chef Menteur), and one at the mouth of the IHNC (Seabrook), that were never constructed. Vast areas of undeveloped, tidal marsh were included in the New Orleans East HPS north of the GIWW and in the Chalmette area south of the MRGO.

Channel and Levee Projects Change the Landscape

Historically, another factor was also at work transforming this landscape. The material dredged from the GIWW and, later, the MRGO, was placed so as to create bands of artificially high ground in the vast expanse of swamps and marshes that protected the eastern margin of the City. Nearly 300 million cubic yards (cy) of earth was moved to construct the MRGO between 1958 and 1968, 60 million more than for the Panama canal. The new high land encouraged local entrepreneurs to think about the navigation alignments as possible boundaries for new forced drainage districts that could eventually be converted into suburban developments.

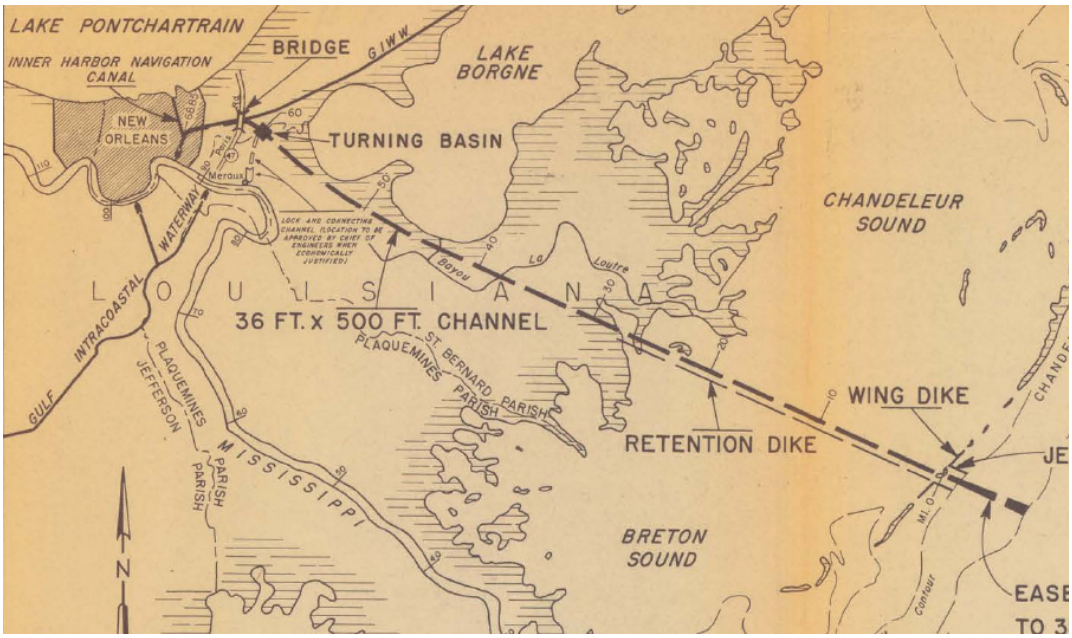


Figure 159. Drawing of the funnel and proposed MRGO route from 1957 GDM 1A, showing a turning basin and spur connection to a new lock into the Mississippi River at Meraux that was never built. The area within City limits east of the IHNC was largely undeveloped except along the river.

The Citrus area of New Orleans East area was subdivided in the 1960s at the same time that the federal levees were being constructed to enclose vast tracts of swamp and marsh. The push to develop then turned into a stampede when Interstate-10 was built with several on-off traffic interchanges in the marsh (Figure 158). So, federal navigation projects led to federal flood protection and then federal highway projects that catalyzed expansion of development into otherwise uninhabitable wetlands.

New Orleans port interests made an economic case for construction of the navigation channels, and they were built first, before anyone thought about the storms or the

environment (Figure 159). The GIWW was dredged during World War II through the wetlands that separated Lake Pontchartrain from Lake Borgne to provide sheltered, submarine-free, inland waterway access for barges and shallow-draft vessels from Mississippi Sound and points east to the Mississippi River via the IHNC lock, built in the early 1920s. It was a relatively small channel 125 ft wide by 12 ft deep, with a design cross-section of 1,400 square feet.

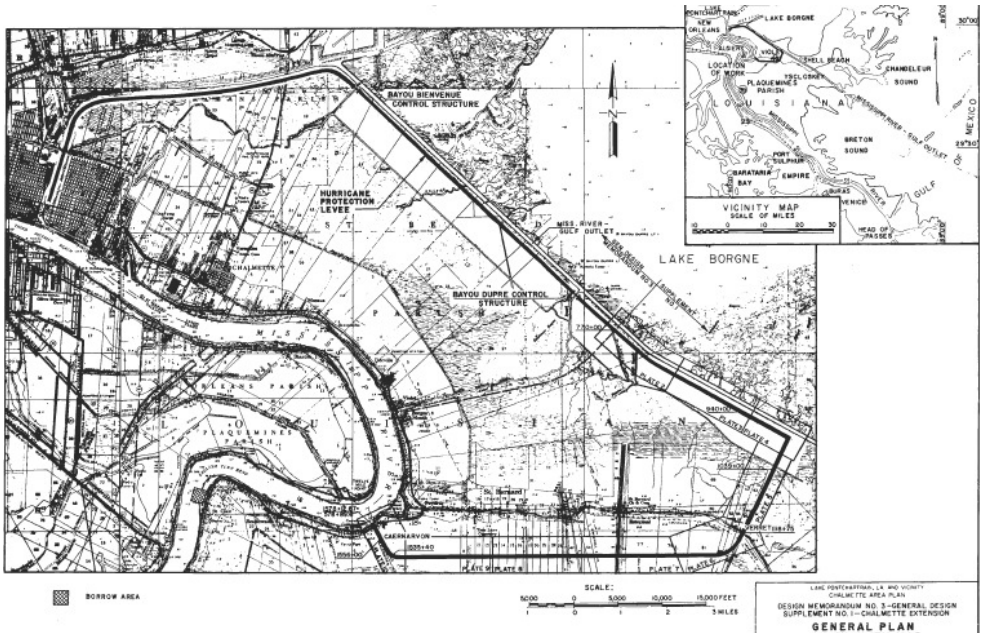


Figure 160. Drawing of MRGO as constructed in 1968 GDM 3, Supplement 1, for the Chalmette HPS, showing the junction with the GIWW farther east of Paris Road than originally proposed (Figure 159). IPET (IPET 2006b, IV-6-9) divided the 76 miles of the MRGO into three reaches. Reach 1 is a 6-mile connection from the MRGO to the IHNC west of the confluence with the GIWW.

Even before the war was over, however, the business community began promoting a far grander project that eventually became the Mississippi River Gulf Outlet (MRGO), a 76 mile long, 40 foot deep, and 500 foot wide ship channel to complement the shallow-draft GIWW (Figure 159). This channel (Figure 160) was planned to have a cross-section of more than 23,000 square feet, sixteen times that of the GIWW (Figure 161). It took more than a decade to marshal support for Congressional authorization, because the channel proposed was expensive and would have to compete with the Mississippi River, a reliable, deep, natural outlet with an already mature port infrastructure serving the same city. After much debate through the 1950s, port interests succeeded in securing local political support and federal funding, but the promise of an economic windfall for the City has never materialized. The MRGO was opened to ocean-going traffic in 1963 but was not fully completed until 1968.

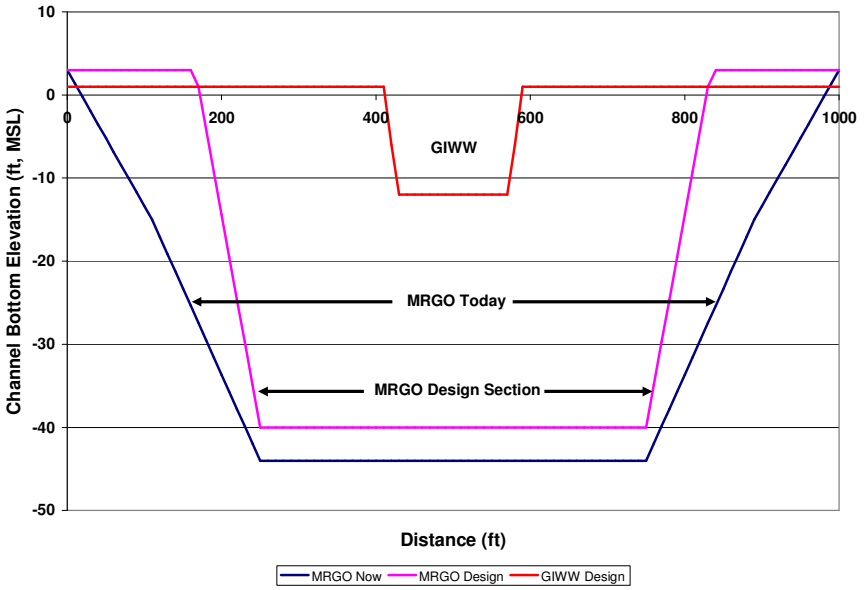


Figure 161. Comparison of GIWW and MRGO cross-sections, as designed and post-Katrina, in Reach 1, the connection to the IHNC. The design cross-section is 23,000 square feet, while the current cross-section averages 29,000 square feet in Reach 1, and 41,000 square feet in Reach 2.

The new channel failed to attract the vessel traffic predicted by boosters, averaging less than 5 inbound and outbound passages per day in a typical year (1998), and carrying only about 3 percent of the New Orleans port freight tonnage (Caffey and Leblanc 1999). On the other hand, it has cost the nation between $10 and $30 million annually for dredging to maintain the authorized channel section, at an estimated price of about $13,000 per ship passage. Most of this dredging has been required for the 40 mile open-water reach between the marsh edge jetties into Breton Sound and the 38 foot contour on the continental shelf that IPET (2006b) has designated Reach 3.

USACE construction of MRGO was destructive to the Breton Sound estuary east of the City. Dredging of the 680 foot channel destroyed more than 10,000 acres of marsh and swamp in the 36 mile reach that IPET (2006b) has called Reach 2. The spoil placed along the south bank buried another 18,000 acres, for a total of nearly 30,000 acres destroyed (Caffey and LeBlanc 1999). Since it was constructed, the channel width has expanded well beyond the design width – up to more than 2,000 ft in places (Figure 162) -- resulting in erosion of an additional 7,000 acres of adjacent marsh (Figure 163).

The full environmental impacts are not limited to the most proximate or obvious. Exclusive of effects in Breton Sound, it includes conversion of 38,000 acres of low salinity to higher salinity wetlands and creation of a 64,000 acre low-oxygen 'dead

zone' in Lake Pontchartrain (LPBF 2006). These are a consequence of increased intrusion of higher salinity waters from Breton Sound into the marsh and, ultimately, Lake Pontchartrain. The north bank of the channel has eroded through the south shoreline of Lake Borgne to facilitate new routes for direct tidal exchange. A significant portion of the wetlands surrounding Lake Borgne that were forced to undergo the ecological succession to more salt-tolerant marsh species did not survive the transition, converting instead to open water. The USACE New Orleans District (NOD) estimated in 1999 that the loss or severe degradation of wetlands attributable to the indirect effects of the MRGO included 1,500 acres of cypress swamps and levee forests, 3,400 acres of fresh and intermediate marsh, 10,000 acres of brackish marsh and 4,200 acres of saline marsh, for a total of 19,100 acres (USACE 1999).

Figure 162. MRGO channel expansion at Bayou Bienvenue crossing near junction with GIWW. GIWW top width generally exceeds 400 ft. MRGO top width ranges from 1,100 to 1,800 ft, more than twice the design width. HPS levee is visible crossing B. Bienvenue. Dredged spoil disposal area behind levee is 3,400 ft wide. Channels accelerated land loss in adjacent marsh, as is indicated by the degree of fragmentation.

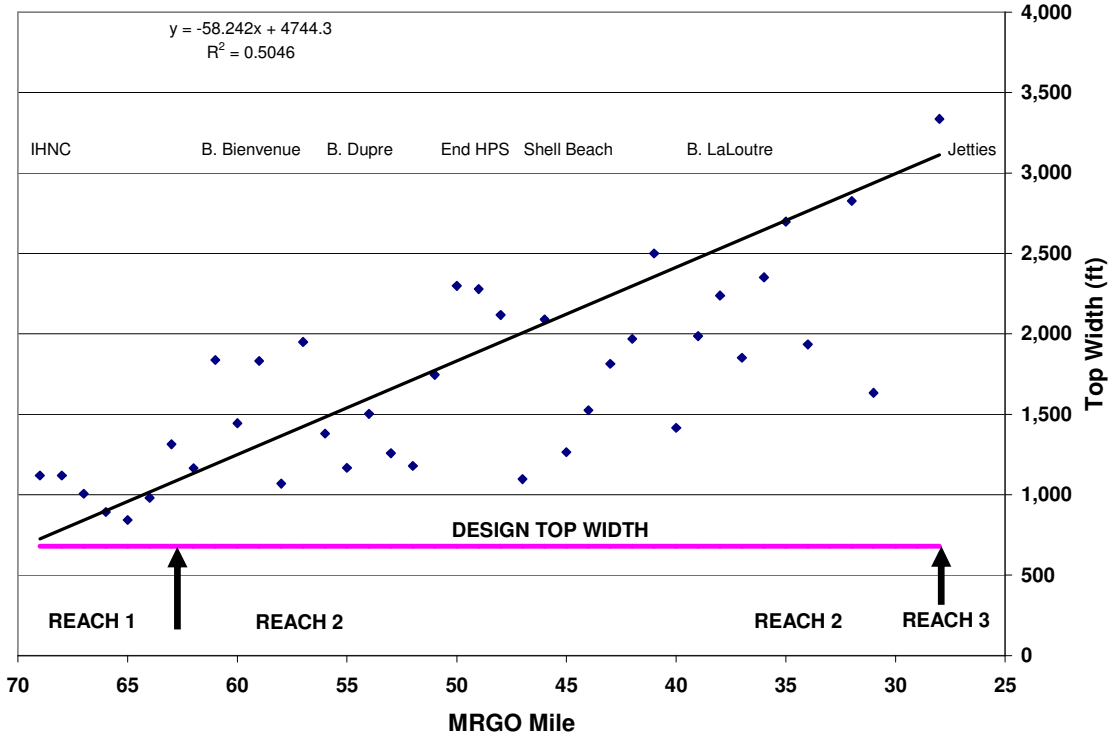


Figure 163. Expansion of MRGO channel by mile from the IHNC on left to jetties at Breton Sound shoreline. Top width is variable but tends to increase toward the coast. Original top width is also shown.

Hydrologically, the MRGO created a short, new pathway for astronomical and storm-driven tides to flow directly from Breton Sound to Lake Borgne and Lake Pontchartrain. The Lake Pontchartrain Basin Foundation, a New Orleans-based environmental organization advocating for the health of the Lake has identified different problems posed to the community by each of the three reaches of the MRGO (Figure 164). The 6-mile connection between the junction of the GIWW and the MRGO terminus in the IHNC has been designated by IPET (2006b) as MRGO Reach 1, and plays the most significant role in transmission of hurricane surge, as is discussed in more detail below. Reach 2, the segment dredged through the marsh, has caused the most environmental degradation, while Reach 3 is the financial albatross, requiring continual dredging to keep the channel open. Since Katrina, at the instigation of Governor Kathleen Blanco and Louisiana's entire congressional delegation, Congress has appropriated funds for a plan to close the MRGO. The 4th Emergency Appropriations Bill (P.L. 109-234) included funds to develop a "comprehensive plan, at full federal expense, to de-authorize deep draft navigation on the Mississippi River-Gulf Outlet." Congress also provided $75 million for the "repair, construction or provision of measures or structures necessary to protect, restore or increase wetlands, to prevent saltwater intrusion or storm surge."

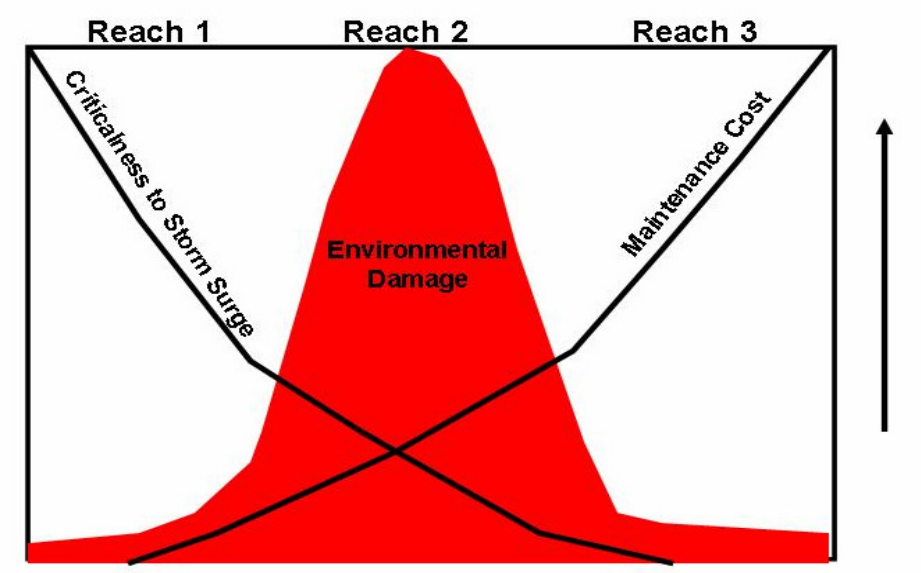


Figure 164. View of the different reaches of the MRGO in the New Orleans community as characterized by the Lake Pontchartrain Basin Foundation. Reach 1 is the 'surge highway.' Reach 2 has caused the greatest environmental damage. Reach 3 is the financial albatross, requiring millions every year in maintenance dredging.

Progress is being made on a consensus plan that may be a final chapter to this saga. The MRGO is now regarded as the equivalent of a Superfund site, in that it may take as much money, and far more time, to remediate the damage than once was expended to build it. It is important to go back in time to understand how the liability was created, and the effect it had on hurricane protection and repeated flooding through New Orleans' "back door" during Betsy, Camille and Katrina.

Environmental and Hydraulics Considerations in Channel Design

We have little information about the design of the GIWW channel when it was rerouted through the wetlands into the IHNC, but can infer that it followed a similar process to that of the MRGO constructed 15 years later. The relevant MRGO GDMs, 1A and 1B, dated 1957 and 1959, respectively, cover MRGO Reach 1 and Reaches 2-3, respectively. Both state that many route alternatives were considered, but no details are provided about anything but the authorized alignment. It is remarkable to a modern reader, given the controversies that have arisen since, how little attention was devoted to any aspects other than the engineering of the channel itself, namely the type of material to be dredged, the spoil areas in which it would be placed, the stability of the cut and expected costs. The effects of this massive channel on the environment of the Lake Borgne estuary and Lake Pontchartrain were addressed in a single paragraph, unsupported by data, as follows.

> "Under existing conditions the salinity in the area traversed by the channel decreases progressively from Chandeleur Sound toward New Orleans, salinities along the Lake Borgne segment being one-fourth to one-third of those in the Sound. With the project in place, salinities in the channel will tend to be higher toward the inner end, particularly so if the channel is effectively separated from the surrounding areas by spoil dikes…Normally, the rainfall over the immediate area and generally over the Pontchartrain Basin tends to make the water conditions fresher, by offsetting the salinity intrusion effects. If the channel is separated by spoil from the surrounding area – except for a few openings – the salinities in the area inland from Shell Beach to the west of the channel will tend to fresher and Lake Borgne should remain essentially unchanged" (GDM 1B 1959, p. 3).

The U.S. Fish and Wildlife Service (USFWS) and the Louisiana Wildlife and Fisheries Commission were apparently unconvinced by the platitudes expressed above. Accordingly, the USACE went a step further to accommodate their concerns about this $90 million project.

> "This agency (USFWS) has expressed concern as to the probable effects the channel will have on the Fish and Wildlife values of the area traversed. The planning is being coordinated with the Service and funds in the amount of $5,000 were made available to the Service in fiscal year 1958 to aid in developing a detail plan that would minimize any probable detrimental effects. The Service prepared an "Interim Report on Fish and Wildlife Resources" …which…concludes that sufficient data are not available upon which to predict the effects of the project and recommends extensive environmental and model studies be made covering a period of several years and costing upwards of $500,000 exclusive of the model. Subsequent conferences and discussions have demonstrated that the model study is impracticable and it has been eliminated…The amount of $64,400 is tentatively made available to the Service in fiscal year 1959 for the studies on a reimbursable basis" (GDM 1B 1959, p. 9).

It is unclear what those studies determined, but based on other statements in the GDM they appear to have been confined to limiting damage caused by dredged material disposal. It is interesting that the recommendation for a hydraulic modeling study was rejected out of hand.

The USACE had constructed a physical model of the Pontchartrain Basin at the Waterways Experiment Station as part of the investigation related to the Barrier Structures planned as part of the GNO HPS, so the USFWS was merely asking that this model be modified to include the effects of the new channel on salinity. The hydraulics of this channel was initially considered either uninteresting or unlikely to affect what would be built. Later, however, while the MRGO was under construction, it appears that enough concern was expressed that this modification to the physical model was made. The modeling study moved forward and a report was issued in 1963 as MRGO construction neared completion (USACE 1963). This study confirmed the worst fears of the environmental agencies, and showed that, in fact, the effects of salinity intrusion would be significant (Figure 165).

During years with relatively low freshwater input to Lake Pontchartrain, the physical model predicted that Lake salinities would increase by a factor of three, and by a greater factor, though with lower overall levels, during a year with high freshwater input. Because the MRGO was already built, the modelers recommended that the barrier structure proposed at the mouth of the IHNC at Seabrook as part of the HPS barrier plan be manipulated to reduce these effects. This structure was never built, and the increases in salinity predicted by the model for Lakes Pontchartrain and Borgne have largely taken place as predicted. Results of more recent numerical model studies (Tate et al. 2002, Carillo et al. 2001, McAnnally and Berger 1997) confirmed that the MRGO has an important role in the propagation of astronomical and moderate meteorological tide into Lake Pontchartrain, and in the flux of higher salinity water into the Lake.

The physical model was never used to model the surge effects of the MRGO. The 1959 GDM covers the whole issue of storm surge hydraulics in concise language that today is seen as a remarkably bald assertion.

> "The influence of the proposed channel on the water surface elevations of the existing lakes and streams connecting therewith inland as far as Highway No. 47 (Paris Road) will be negligible…Lake Borgne, with its large openings into Mississippi Sound and the Gulf, controls the elevation in all of the streams inside of Bayou La Loutre crossing, and the small additional area of the project channel will not have any material influence on the elevations produced by ordinary tides, including those from lesser storms. For major storms and hurricanes when tides roll across the marsh many feet deep, as well as through the

open water connections, the effect of the new channel will be of no consequence (GDM 1B 1959, p. 3).

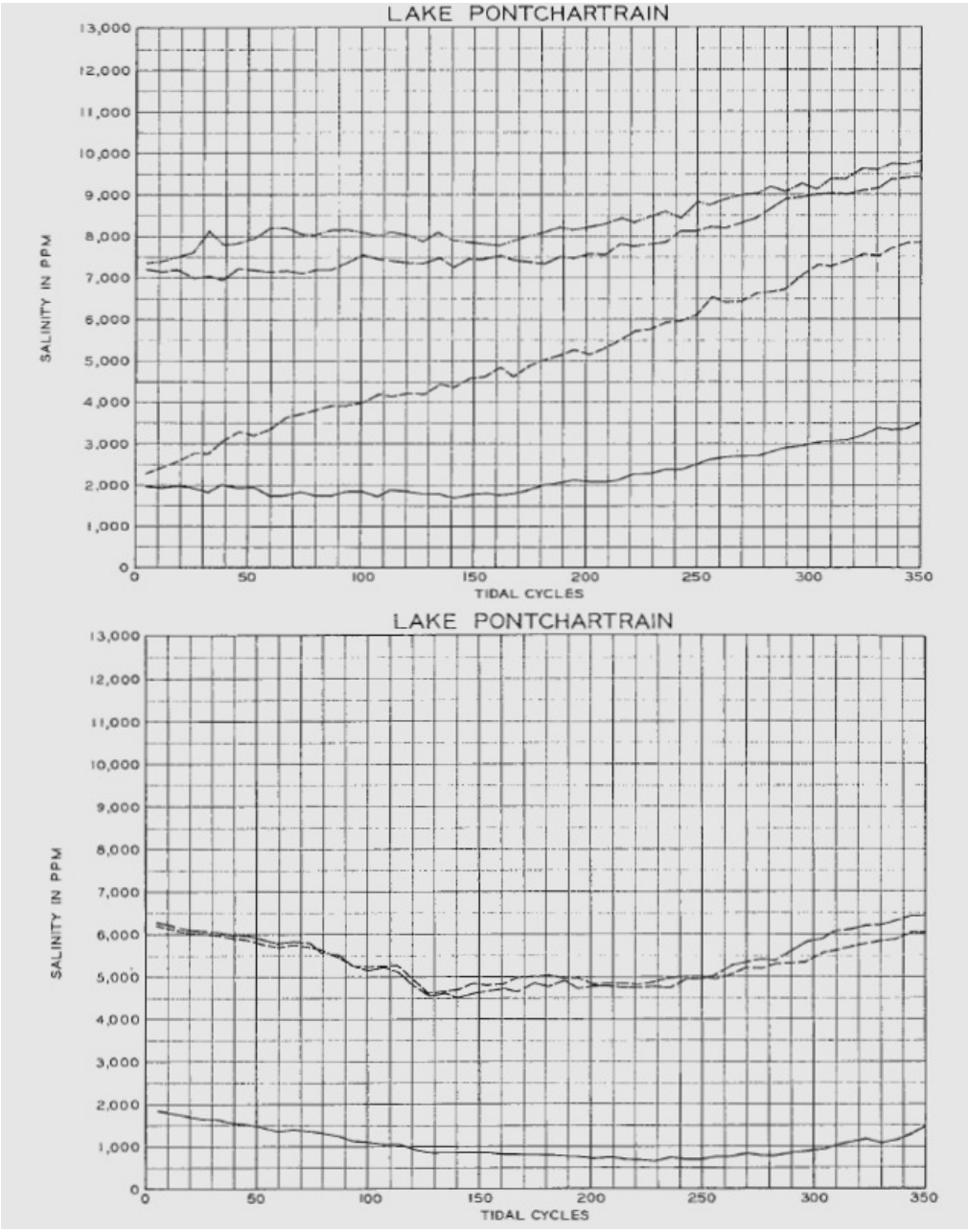


Figure 165. WES Physical model results showing predicted effect of the MRGO on salinities in Lake Pontchartrain in a low freshwater inflow year (top) and high inflow (bottom). Top lines in each panel represent Lake salinity with MRGO and bottom lines without. The MRGO is predicted to raise Lake Pontchartrain salinity by 4,000 to 6,000 parts per million (6 parts per thousand) in both cases (USACE 1963).

More attention was paid in the GDM to the potential for storm wave propagation up the MRGO (deemed negligible), but any serious analysis of the hydraulics would have to wait until after Hurricane Betsy, when the channel was already a *fait accompli*. The post-Betsy study by Bretschneider and Collins (1966) will be discussed below.

A final point concerns the stability of the channel cut. The channel was to be dredged through relatively unconsolidated Holocene materials, primarily clays, but including an almost ubiquitous 10 feet of marsh/swamp peat at the surface and scattered deposits of sandy-silt and silty-sand at greater depths within the cut (Figure 166). The channel cut was designed with 1V:2H side-slopes. It was not expected they would stay that way very long. Concern about channel expansion appears to have been limited, however, as is seen in the GDM discussion of the need for channel protection below.

> "No channel protection is recommended initially; however, erosion due to wave wash in open areas can be expected in the upper part of the channel slope where the peat and highly organic clays are exposed. Protection for this area can be provided if and when the need for it becomes necessary. No channel protection is included in the overall cost estimate of the project. It is presumed that sufficient rights-of-way will be furnished by local interests to preclude use of channel protection, or that additional rights-of-way will be furnished if the need arises" (GDM 1B 1959, p. 5).

Despite the limited vessel traffic, expansion of the channel has been rapid, between 10 and 20 feet per year on each bank, such that the top width of the channel now exceeds 2,000 feet in some reaches (Figure 163). This has necessitated placement of limestone boulders on the south bank where the dredged material was discharged, and upon which the Chalmette hurricane protection levee was later built, to protect the stability of this structure. Until recently, the north bank was left unprotected, resulting in several openings through the shoreline of Lake Borgne. Substantial sums have been spent in recent years under the joint State-USACE coastal restoration program that was authorized by the Coastal Wetlands Planning, Protection and Restoration Act of 1990 (CWPPRA), also called the "Breaux Bill," in a futile effort to stop MRGO expansion. Greater expenditures are now planned for the same purpose in the post-Katrina context (Figure 167).

The narrow focus on facilitating economic development and lack of concern for the environment evidenced in the MRGO GDMs – not atypical of other USACE projects of the time -- helped make the case in 1969 for passage of the National Environmental

Policy Act (NEPA). NEPA required federal agencies to develop environmental impact statements and to engage the public more substantively in the development of major federal projects. In the early 1980s, when the USACE was ordered to retroactively comply with NEPA to justify elements of the GNO HPS, specifically the barrier structures at the Lake Pontchartrain passes (Figure 158), the NOD found this task sufficiently intractable that they changed the plan to eliminate these features. We will discuss in the next chapter how this switch from the inconvenient "barrier plan" to the "high-level plan" in 1985 set in motion a shift in the design process that ultimately led to the failure of the Orleans Metro drainage canal I-walls during Katrina.

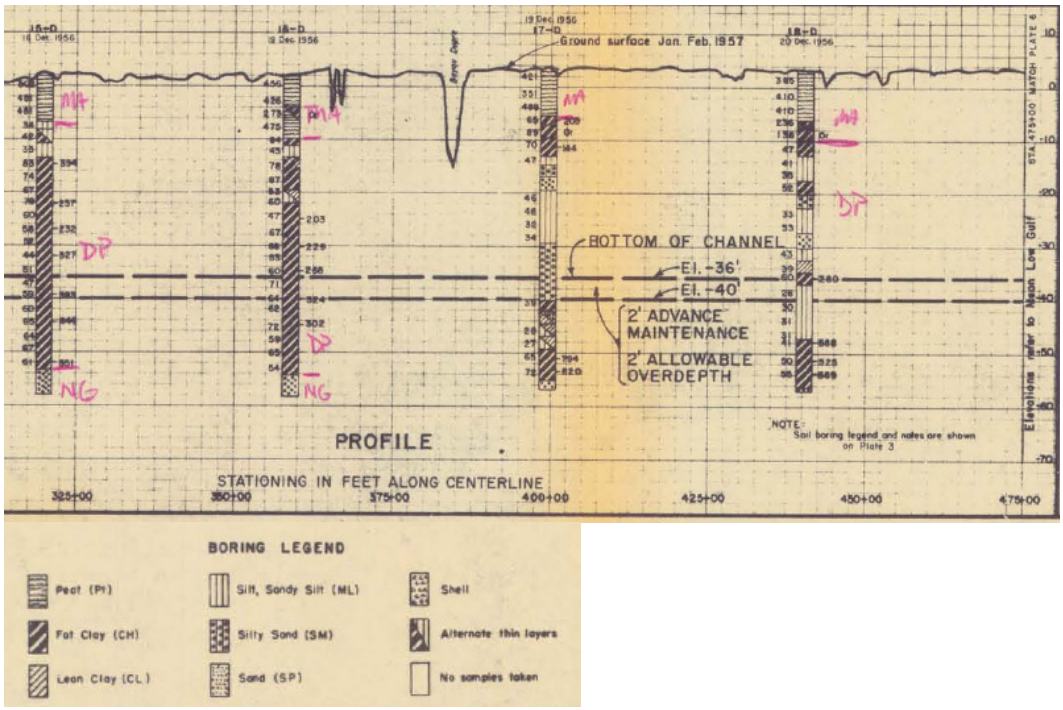


Figure 166. Soil borings along the MRGO channel alignment in the vicinity of the Bayou Dupre crossing from 1959 GDM 1B, showing 10 feet of marsh/swamp peat at the surface and silty, sandy strata at greater depths.

Storm Surge Dynamics and the Funnel

Damaging storm surges propagated into the IHNC from Lake Borgne through the MRGO channel reach west of the junction with the GIWW in 1965, 1969 and 2005, during hurricanes Betsy, Camille and Katrina, respectively. These were all storms that generated higher surges in Lake Borgne than in Lake Pontchartrain, and caused flooding in New Orleans through the MRGO "back door" (Figure 168). The convergence of the two navigation channels, and the levee systems built along their banks, created what has widely been described as a 'funnel,' with the mouth open to

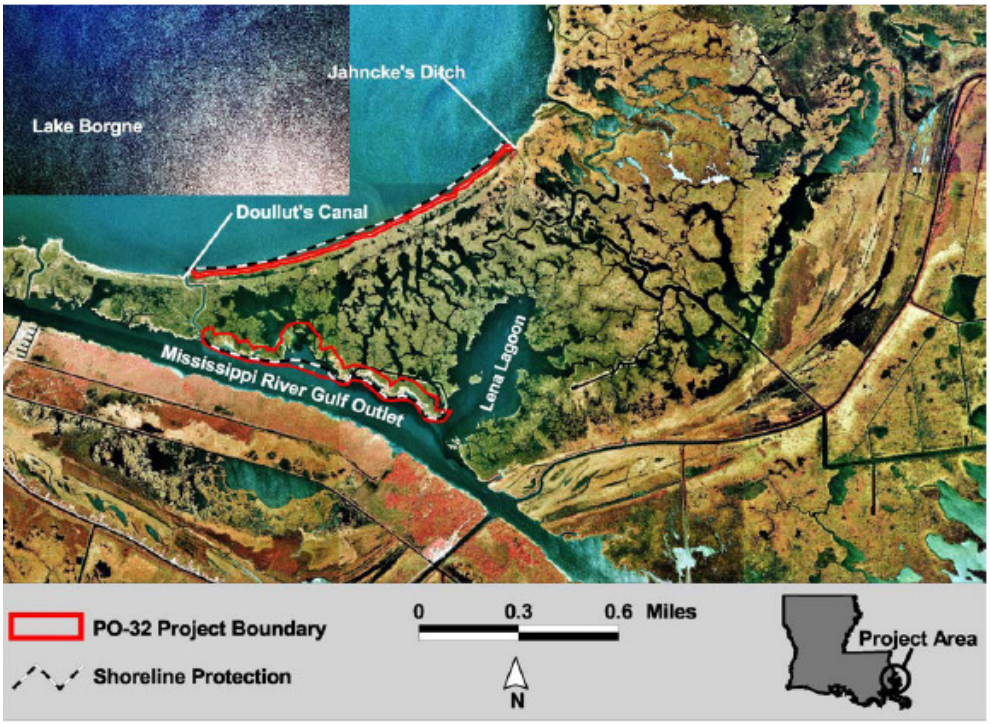


Figure 167. Wetland restoration project to mitigate damage caused by the MRGO funded by a federal-state partnership under the Coastal Wetlands Planning, Protection and Restoration Act (CWPPRA).

Lake Borgne on the east, that favors the amplification of surges generated by cyclones tracking over the city or to the east (Figure 158). The throat of this funnel is the 6 mile portion of the MRGO channel that was constructed inside the original GIWW right-of-way (Figure 169) that IPET has referred to as the "hurricane highway" (IPET 2006b, I-6) or "MRGO Reach 1" (Figure 160).

Hurricane-generated surge arises from the interaction between wind-forced water flow and the geometry of the receiving coastal landscape. Like a tsunami, the elevation of the surge is trivial in deep water but rises up against any barrier, whether an island, beach, floodwall or levee, particularly if there is no ready avenue for escape. In coastal Louisiana, the hurricane protection and river levees are the highest features in the landscape, so this is where the surge rises to its greatest heights. At the same time, dredged channels, far deeper than any natural waterway, convey the surge inland particularly if they run normal to the coast. Adjacent wetlands also convey surge, but provide far greater resistance and a shallower depth of flow.

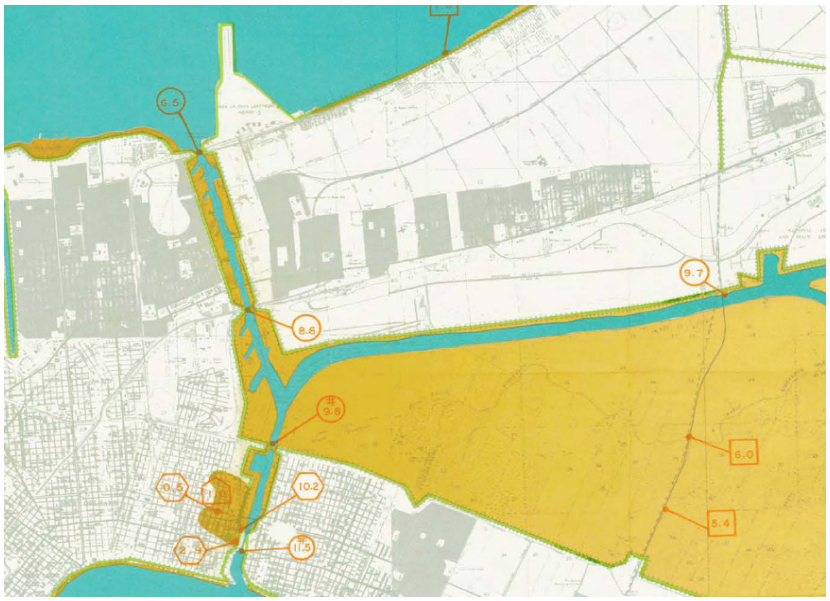


Figure 168. High water caused by storm surge of Hurricane Camille on August 17, 1969. Peak elevation was 9.7 ft (NGVD29) at Paris Road, 10.2 ft on the north side of the IHNC lock, and 6.5 ft at the IHNC opening into Lake Pontchartrain. New Orleans flooding was limited to a small area of the Upper 9th Ward on the west side of the IHNC. New Orleans experienced a Lake Borgne/MRGO surge during Camille, while Lake Pontchartrain levels remained relatively low.

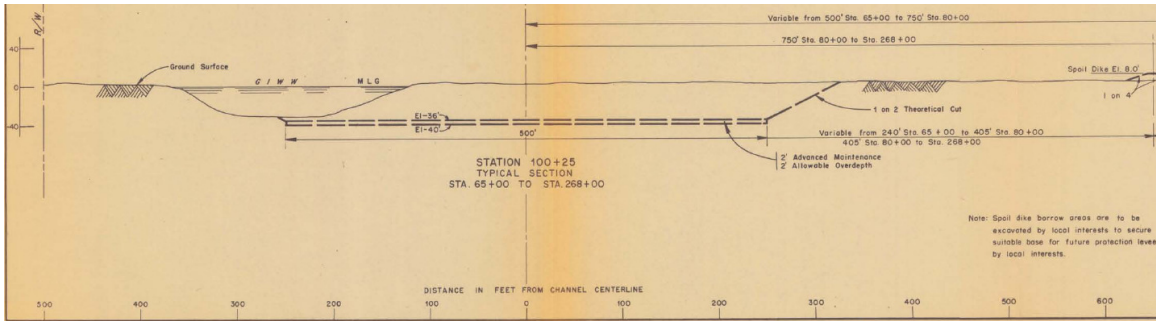


Figure 169. Design drawing for enlargement of the GIWW to accommodate MRGO ship traffic in MRGO Reach 1 west of Paris Road from 1957 GDM 1A. Note that the GIWW in this reach had scoured to far deeper than the 12 ft authorized depth prior to its incorporation into the MRGO.

While the throat of the funnel, MRGO Reach 1, is a large navigation channel (44 by 1,000 ft), it is small relative to the gap (7.5 miles) so that water pushed into the jaws tends to pile up as can be seen in an ADCIRC simulation of the surge from Hurricane Betsy (Figure 170). IPET describes MRGO Reach 1 with respect to storm surge as follows.

> "The critical section of the MRGO is Reach 1, the combined GIWW/MRGO. It is through this section of channel that Lake

> Pontchartrain and Lake Borgne are hydraulically connected to one another via the IHNC. Reach 1 existed as the GIWW prior to the construction of the MRGO, although the maintained depth was lower. Because of this connectivity, the local storm surge and astronomical tide in the IHNC and in the section designated GIWW/MRGO is influenced by the tide and storm surge in both Lake Pontchartrain and Lake Borgne…The Reach 1 GIWW/MRGO section is very important in determining the magnitude of the storm surge that reaches the IHNC from Lake Borgne and Breton Sound. If the hydraulic connectivity between Lake Pontchartrain and Lake Borgne is eliminated at a point within this section of channel, tide or surge to the west of this point will become primarily influenced by conditions in Lake Pontchartrain" (IPET 2006b, IV-6-2).

The term "critical section" as used by the IPET surge modeling team refers to that portion of the channel in which hydraulic gradients are steepest and the velocity increases as a result of constriction. The highest mean velocities observed in deep rivers and dredged channels like the MRGO are generally constrained by turbulent and inertial forces to 10 feet per second (fps) or less, which is far lower than the classic transition from critical to supercritical flow (Froude number > 1) for a channel of the depth of the MRGO. But conveyance or discharge through a given channel cross-section is still limited by this constraint.

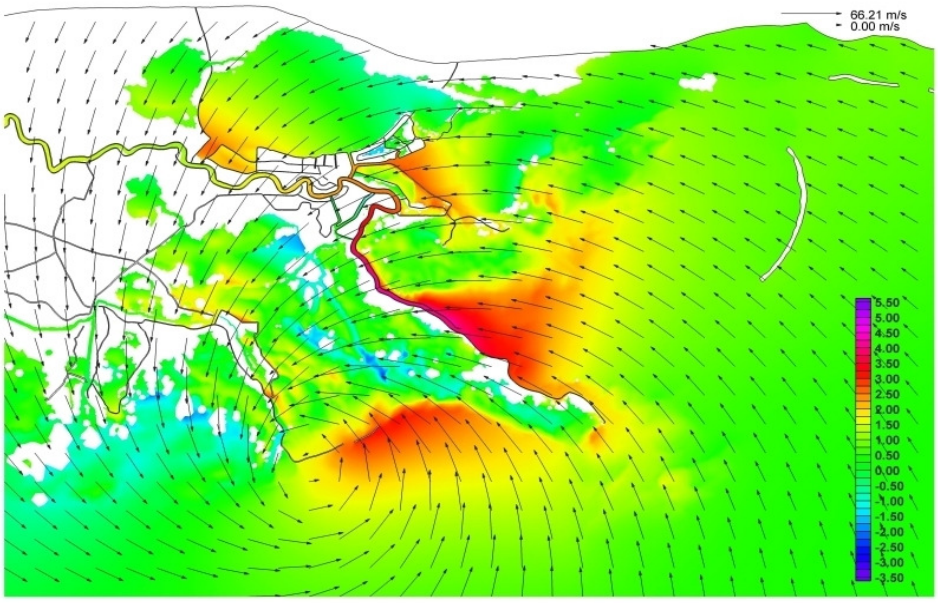


Figure 170. Surge build-up in the funnel and against the east bank levees of the Mississippi River in a snapshot from an ADCIRC simulation of Hurricane Betsy by Joannes Westerink at the

University of Notre Dame (http://www.nd.edu/~adcirc/betsy.htm). Eye of the storm is just making landfall at Grand Isle. Surge elevation is given in meters on scale at right (1m=3.28 feet).

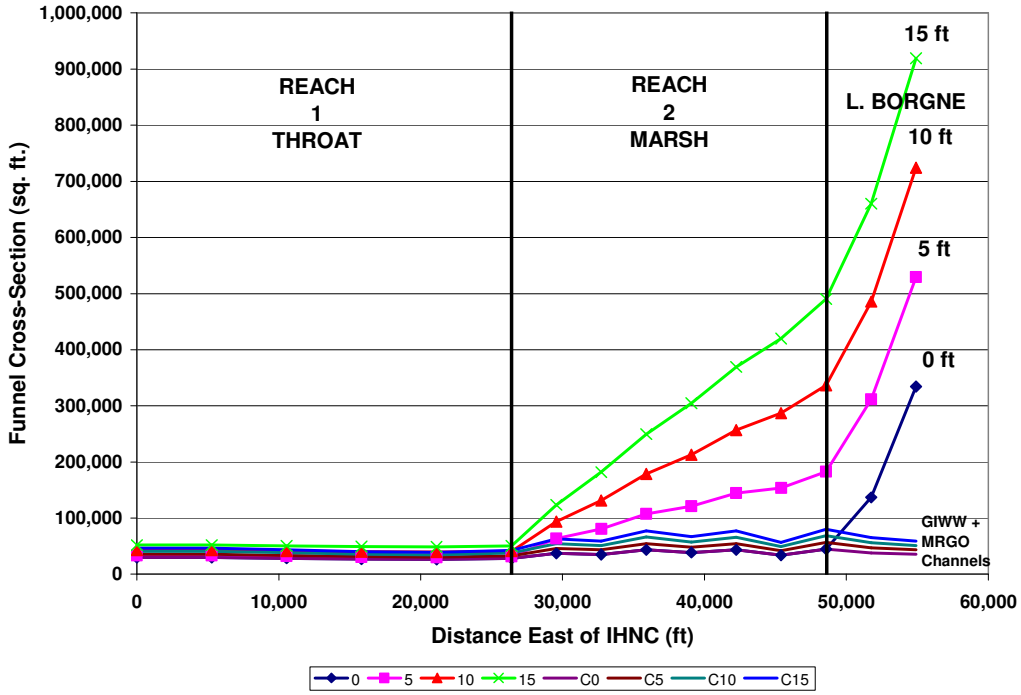


Figure 171. Cross-section of the funnel between the New Orleans East and Chalmette (St. Bernard) HPS levees at surge levels from 0 to 15 ft MSL, showing constriction in MRGO Reach 1.

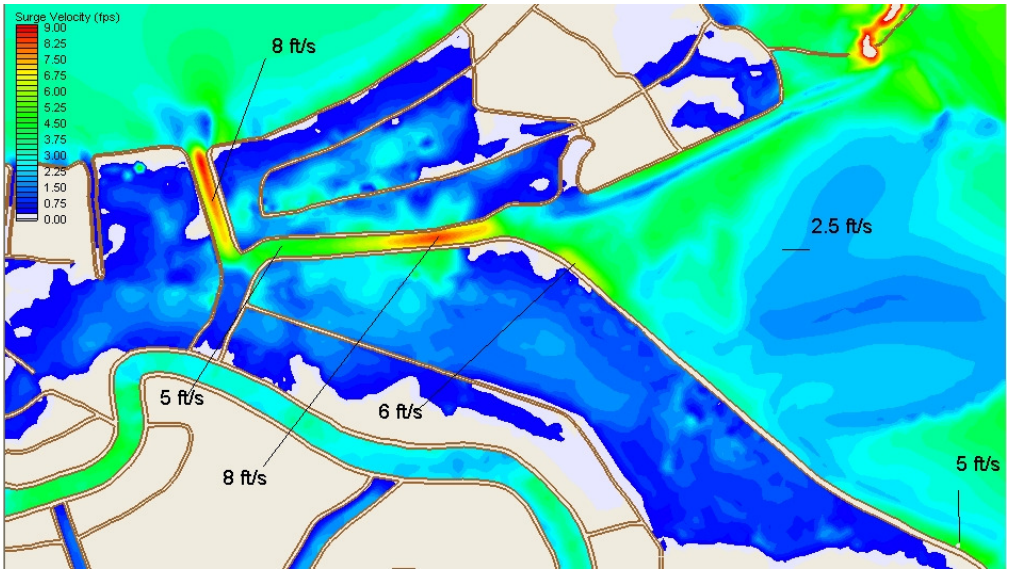


Figure 172. Peak flow velocities in the funnel in Katrina ADCIRC. The red regions are zones of highest velocity (>8 fps). MRGO Reach 1 and the northern reach of the IHNC are 'critical,' where a reduction in cross-section has the most effect on surge propagation. High velocities also occur in the MRGO at both ends of the zone of maximum surge (Figure 8.16), while low velocities occur in the GIWW and at the south end of the IHNC.

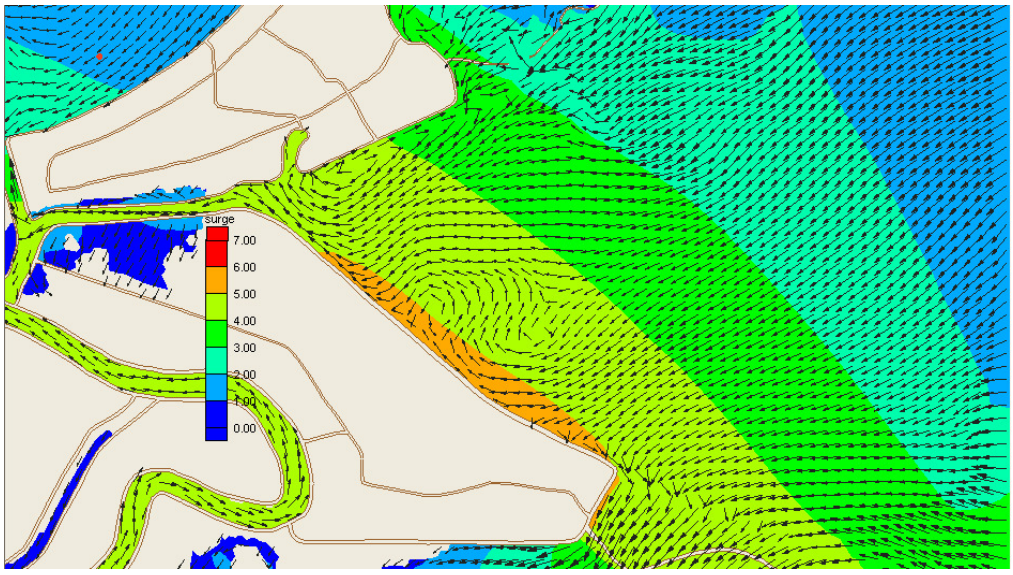


Figure 173. Flow direction in the funnel at peak surge in Katrina ADCIRC simulation. The surge reached more than 5 meters (15 ft) on the MRGO hurricane protection levee. Flow converges from all directions toward the throat of the funnel, but some that cannot be accommodated moves east as a return flow in the GIWW. The MRGO conveys flow away from the zone of maximum surge along the HPS levee that follows Reach 2.

In the 1,800 foot narrows between the New Orleans East and St. Bernard hurricane levee systems in MRGO Reach 1, the cross-section available rises more slowly with the hurricane surge than the rate at which water enters the eastern end, so velocity rises (Figure 174). If we look at the maximum depth-averaged velocities predicted by ADCIRC throughout the funnel area during Katrina, the highest values occurred at the eastern end of MRGO/GIWW Reach 1 and in the northern reach of the IHNC at inlet constrictions (Figure 172). At the time that the surge is highest against the hurricane protection levee along the south bank of the MRGO in Reach 2, flow throughout the funnel converges on the throat, but cannot be fully accommodated, so that a return flow is predicted to the northeast in the GIWW (Figure 173). Flow also moves away from the peak surge zone out both ends of MRGO Reach 2.

Investigations Following Hurricane Betsy

Hurricane Betsy made landfall as what we would now call a Category 3 storm near Grand Isle during the last hours of September 9, 1965, and tracked south of the City following the west bank of the Mississippi (Figure 174). It caused disastrous flooding in New Orleans East, the Lower 9th Ward, parts of St. Bernard and in eastern parts of the Orleans Metro area (Figure 175). Flooding depths in parts of the Lower 9th Ward, and in several neighborhoods west of the IHNC and south of the Gentilly ridge

reached up to 8 feet (Figure 176). Flooding resulted from overtopping and breaches of levees and floodwalls along the IHNC and GIWW, and the 40 Arpent Levee in St. Bernard (USACE 1965). Flooding in New Orleans and St. Bernard resulted from a surge that rose in Lake Borgne. This is illustrated by a comparison of gage data from the MRGO at Paris Road and the IHNC at Seabrook near the opening to Lake Pontchartrain. The MRGO at Paris Road reached 10 feet (NGVD29), while the IHNC near the Lake peaked 4 feet lower, at about 6 feet (Figure 177).

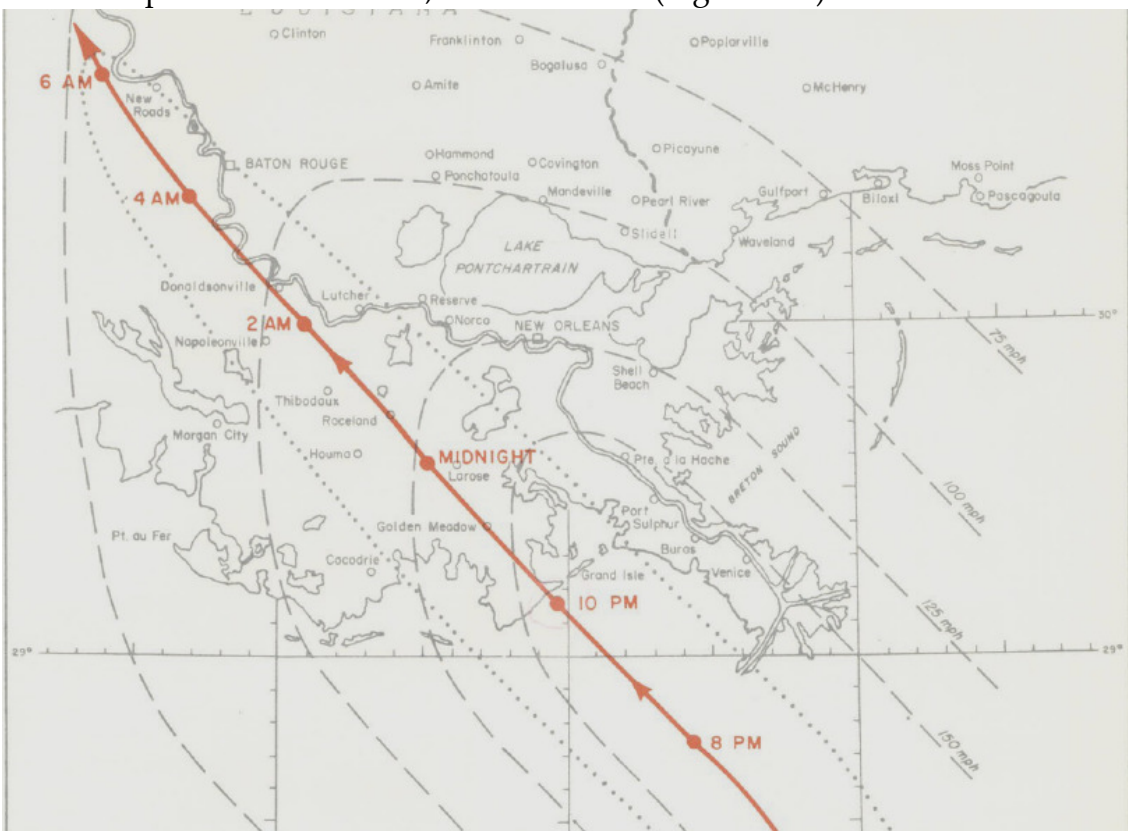


Figure 174. Track and estimated maximum winds of Hurricane Betsy, September 8-9, 1965 from USACE Hurricane Betsy report (USACE 1965).

It is ironic that flooding through the MRGO provided the impetus for funding the GNO HPS, a system that was planned before the MRGO was built, and was primarily oriented toward protecting the City from a Lake Pontchartrain surge. There was no gage in the IHNC except the one at Seabrook, but high water marks in the IHNC were up to 10 feet (NGVD29) opposite the MRGO junction and south toward the lock, the same as the Paris Road gage peak (USACE 1965). Floodwall and levee breaches on the IHNC occurred in essentially the same locations, albeit on lower structures, as they did 40 years later during Katrina.

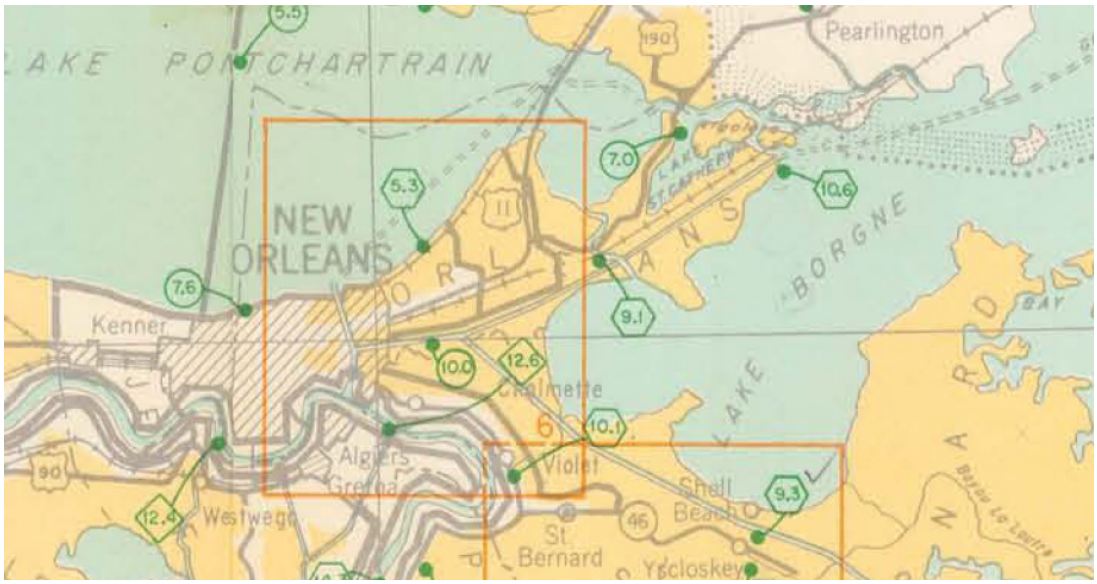


Figure 175. High water levels (ft, NGVD29) outside the HPS and flooding in New Orleans caused by Hurricane Betsy (USACE 1965).

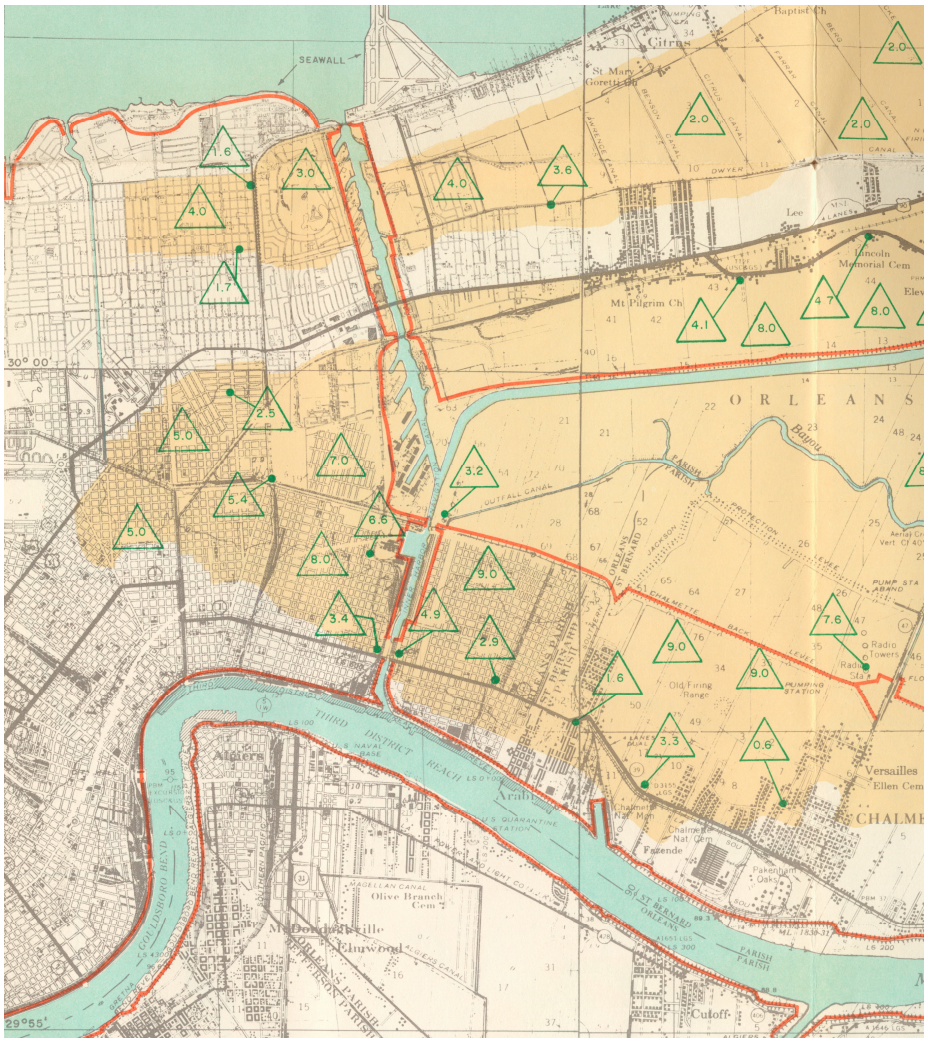


Figure 176. Flooding depths within New Orleans and St. Bernard caused by Hurricane Betsy ranged up to 9 feet.

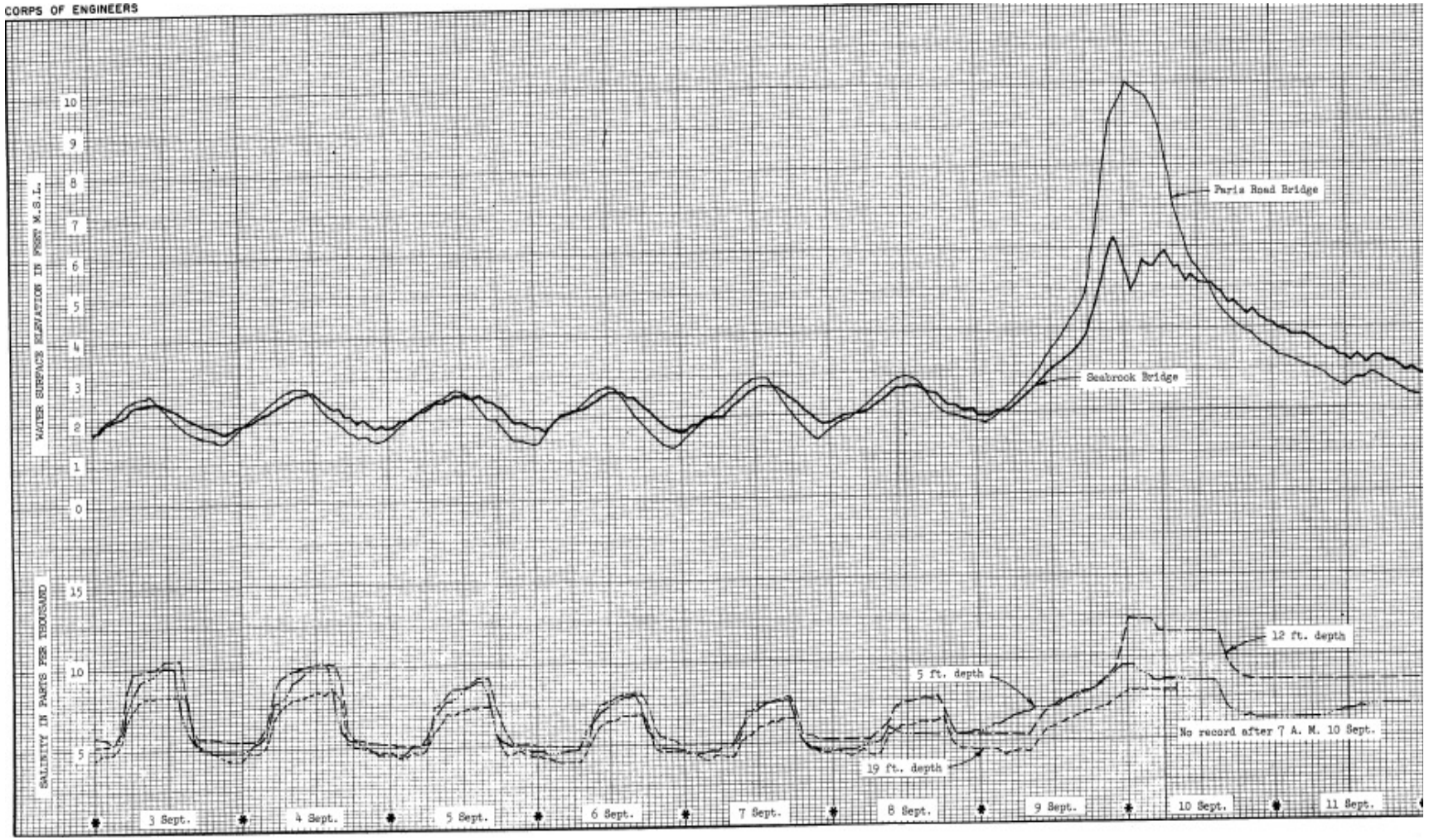


Figure 177. Gage records during Hurricane Betsy from the MRGO at Paris Road Bridge and from the northern end of the IHNC at Seabrook where it enters Lake Pontchartrain (USACE 1965). Notice that the normal tide level prior to any storm effects is shown as fluctuating around +2 "M.S.L.," a value consistent with the 1965 NGVD29 datum. Maximum surge at the southern end of the IHNC during Betsy was +10 feet, essentially the same as at the Paris Road bridge.

The damage caused by Hurricane Betsy led those harmed to blame the MRGO, and lawsuits were filed against the USACE. At this point, the USACE felt compelled to commission the first rigorous analysis of the role of the channel in storm surge conveyance. Consultants Charles Bretschneider and J. Ian Collins of the National Engineering Science Company issued their report in September 1966, a year after Betsy struck (Bretschneider and Collins 1966). They used early one-dimensional computer modeling techniques to evaluate "the effects, in the vicinity of the Inner Harbor Navigation Canal, due to rapidly and slowly rising surges… for four cases." These cases were:

I. Existing levees with no MRGO
II. Existing levees with MRGO
III. Proposed levees with MRGO
IV. Proposed levees with no MRGO

The GNO HPS was in the first year following authorization, so the hurricane levee on the MRGO spoil bank was still "proposed," although the 6 to 8 feet of spoil had some effect during Betsy. The GIWW and MRGO were in place, but the "throat" of the funnel was not as constrained as it is today (Figure 178).

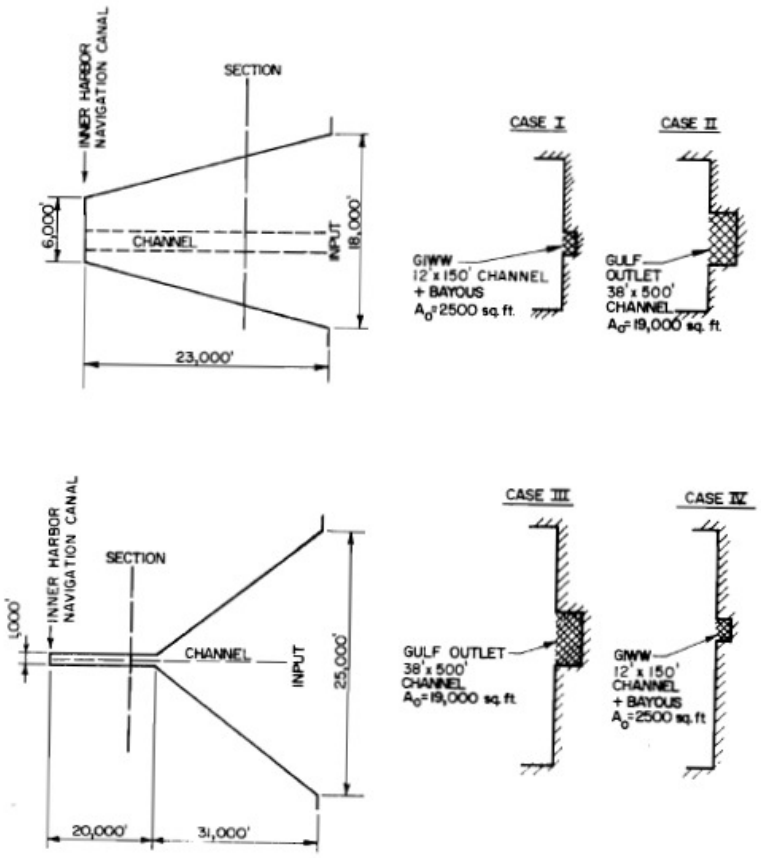


Figure 178. Diagrams showing the geometry used by Bretschneider and Collins (1966) for simulation of surge in the funnel under the four test conditions.

The geometry was highly schematized, and the model was constrained by a number of empirical tuning coefficients that limit the physics in ways that would not be accepted today. Partly because the set-up was so simple, however, the results, if considered qualitative and representative rather than truly quantitative, can provide important insights into how storm surge propagates through this system. The first of these, apparent in the choice of cases to be analyzed, is that the enlarged channel and the geometry of the levee system will both produce effects on surge that can be distinguished and treated somewhat separately. The second was the recognition of the time-dependency of the surge produced by a moving storm. This means that surge can propagate only a limited distance in a given direction if the winds that are forcing it continually change in direction and magnitude, so a steady-state condition cannot be assumed. Finally, Bretschneider and Collins (1966) made the first serious effort to integrate wetlands into a surge analysis, and conducted an analysis of the degree of resistance to flow that a marsh might impose, compared to a deep channel. They concluded that the resistance would be more than three times greater for flow over the marsh than through the channel.

> "It must be expected that a large channel cut through marsh areas will permit more water to arrive at a faster rate in the interior of the marshland, at least in the immediate vicinity of the channel. In addition, the maximum elevation and the steady state peak will be reached at an earlier time….Without the channel, the water will rise over the marshlands at a slower rate and it will take a longer duration to reach maximum elevation and steady state conditions. Similarly, after the storm has passed it will take longer for the surge to recede since now there is no channel to act as a drain…The velocity of flow through the channel will be two to four times as great as that over the marshland, but the volume of water (velocity times cross-sectional area) determines the total amount of water which will enter…It is this later factor which tends to cause an increase in surge because of the Mississippi River-Gulf Outlet."

Bretschneider and Collins (1966) calculated surge along two traverses through the funnel for Hurricane Betsy (Figure 7.22).

Results showed a dramatic increase in predicted surge moving inland through the funnel along each of these traverses, from a maximum of 10 feet MSL at the coast to about 16 feet in Lake Borgne (Figure 180). The actual surge was not known at the

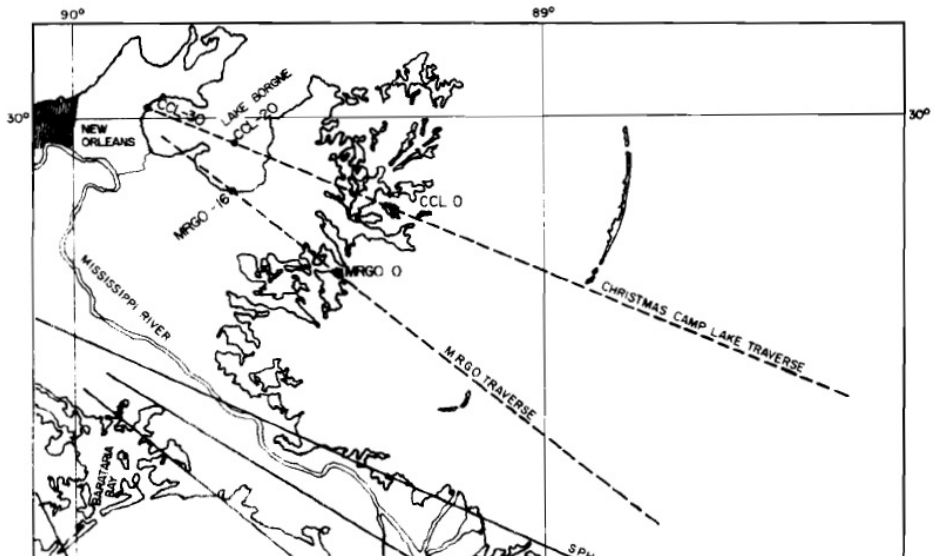


Figure 179. Test traverses along which surge was computed by Bretschneider and Collins (1966) for Hurricane Betsy and for the synthetic critical SPH storms. MRGO 0 and CCL 0 are at the marsh shoreline of Breton Sound, while MRGO -16 is at Shell Beach and CCL – 30 is at the Chef Menteur Pass.

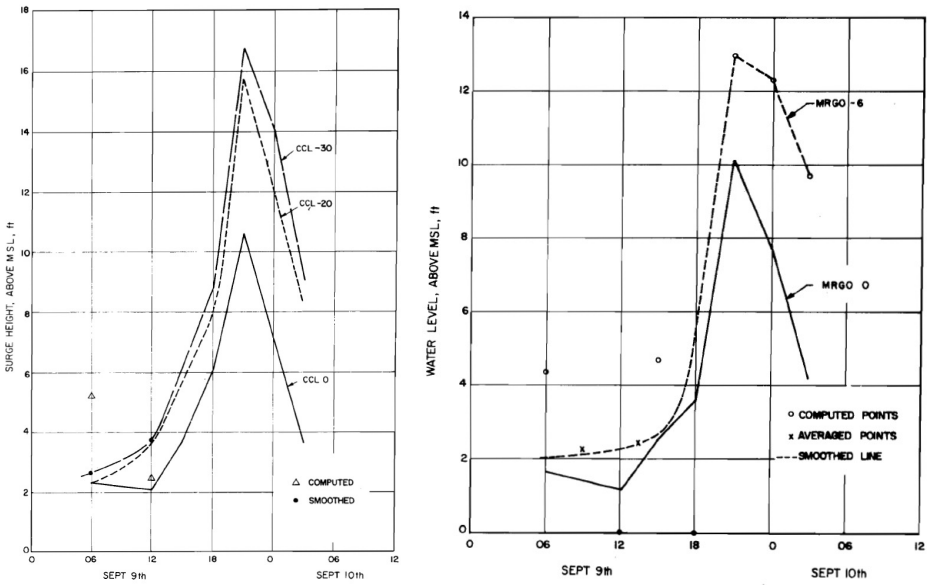


Figure 180. Surge elevations along Christmas Camp Lake Traverse (left) and Mississippi River – Gulf Outlet Traverse (right) computed using Bretschneider and Collins (1966) model. MRGO -6 in the right panel should read MRGO -16 and corresponds to the gage location at Shell Beach (Alluvial City).

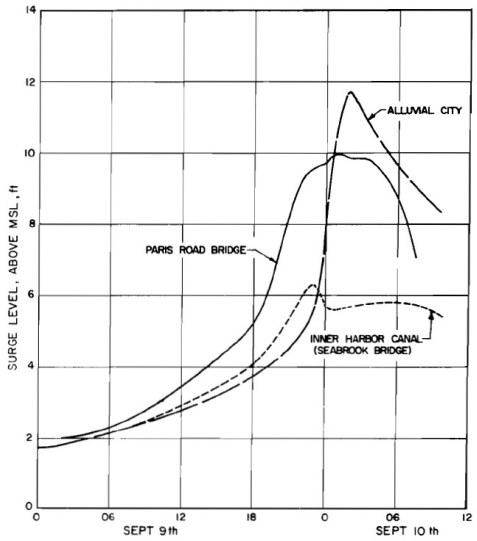


Figure 181. Computed maximum at MRGO -16 corresponds reasonably well to the observed gage data at Shell Beach (Alluvial City) but the computed value at CCL -30 is much higher than the observed value at the Paris Road bridge (Bretschneider and Collins 1966). Note the much lower surge at Lake Pontchartrain in the IHNC.

coast and had to be developed by the model. Computed maxima correlated reasonably well with available gage data at Shell Beach (MRGO -16). The computed values for Lake Borgne (CCL -20, CCL -30) were, however, significantly higher than what was observed at the Paris Road gage five miles to the west (Figure 184). The model also predicted that the surge peak would arrive at the gages about 4 hours earlier than it actually did.

Bretschneider and Collins (1966) attributed the error in the timing of the surge maxima to a lack of consideration of inertia effects in the storm tide equation used. For the error associated with the level of the maxima, they stated that "no system of equations can really be expected to predict, with a great degree of accuracy, the complex physical phenomenon of flooding over marshland, bayous, houses, trees, etc." They concluded that "the computation of storm surges over semidry land must be regarded as an art rather than a science."

Bretschneider and Collins (1966) failed to mention far more likely sources of error. Later surge modelers have found that (1) correct parameterization of the time-dependent forcing wind stresses, and (2) accurate topography and bathymetry are the most critical factors needed for realistic surge simulation. At the time that this work was done, the dynamics of winds for any particular storm like Betsy were poorly understood and were essentially unknown both offshore and inland in the area of

interest. With respect to the highly schematized geometry employed, Bretschneider and Collins (1966) did not challenge the belief prevalent at the time that mean sea level was equivalent to zero NGVD29, the datum at the few gages operating in the area of interest (Figure 177). Further, they state that the "marsh was assumed to have a water depth of -2 feet, mean low water." As a result, the intertidal marsh surface, normally between 1 and 2 feet above mean sea level, was represented as being 3 to 4 feet lower, well below mean sea level.

A reasonable modern interpretation of Bretschneider and Collins (1966) results is that the Hurricane Betsy wind fields used to develop the surge were not accurate either over the shelf, over the funnel, or both. They do say that "in most practical cases of hurricanes, the assumption of constant wind speed and direction over the continental shelf is not justified." But, instead of challenging the wind and geometry data they were given by the USACE, and pointing out the need for better information to improve results, the consultants assumed that the sketchy information provided was entirely accurate, and then used a statistical curve matching approach, together with empirical "surge adjustment factors" and "planform factors" proposed by the New Orleans District, to bring their model predictions into alignment with the observed values at Paris Road bridge. They do express some dissatisfaction with this approach, which has the effect of subordinating their elegant physical scheme to empirically derived factors with no physical meaning.

> "Confidence in such a prediction scheme could be expected for a hurricane which has a similar traverse and speed to Betsy. It is apparent that more elaborate prediction schemes can be developed. Because of their empirical nature, prediction schemes can only be expected to give reliable results for conditions which are almost repetitive. In particular, for example, any hurricane whose eye passes over one of the traverses will yield too low a storm surge prediction along that particular traverse because the resulting prediction equation is not applicable in this case…For conditions of sparse data and several completely different hurricane tracks the method appears to be impractical, and it was therefore necessary to return to the "surge adjustment factor" based on matching peak surge."

The Bretschneider and Collins (1966) model, when the physics were unconstrained by empirical tuning factors, showed the expected increase in surge due to the "funnel" effect. They were forced to abandon this approach, however, when they were apparently pressed to develop an estimate of the relative contribution to the surge of flow through channels and over the marsh. To do this, they retreated to a steady-state

assumption that is inherently at odds with actual storm surge dynamics. The steady-state assumption results in an exact balancing of forces that removes or disregards the spatial and temporal variations in velocity that govern surge generation. It is not surprising then, that the residual influence of the MRGO channel "had an effect of increasing the storm surge throughout the marshland for Hurricane Betsy by about 0.3 to 0.4 feet maximum." Although the methodology is suspect, it is important to note that Bretschneider and Collins (1966) conclude that the MRGO did have an effect on surge, albeit a small one during Betsy, which would not have been considered by the designers of the GNO HPS nearly a decade earlier.

In thinking about the generalizability of this result to other storms, it should be pointed out here that Bretschneider and Collins (1966) used a diminutive 19,000 square foot channel cross-section for the MRGO that corresponds to the dimensions of the maintained navigation channel, rather than the size of the dredged cut (Figure 178). That channel now averages 29,000 square feet in Reach 1 and 41,000 square feet in Reach 2 when the water is at mean sea level (Figure 161). And of course, the marsh was represented as a large lake with a bed 3 to 4 feet below the actual elevation, so this would have worked to increase the flow over the marsh at the expense of the channel. Given that the channel is twice the size modeled, and that the marsh has a much higher elevation than modeled, it is expected that the actual effect of the MRGO on surge during Betsy might have been greater than the consultants reported by a factor of 4 to 6. An increase in surge associated with Betsy of 1.2 to 2.4 feet over the expected elevation should have been of interest to the levee designers, but the conclusions of this report tended to reduce rather than raise concern.

The most interesting analysis carried out by Bretschneider and Collins (1966), and the only part that withstands modern scrutiny, pertains to the effect of the levee alignments and channel in the vicinity of Reach 1 on surge in the IHNC. This was a simpler problem than the others that the consultants tackled, because it essentially reduced to one of channel and overbank conveyance in Reach 1. They presented results for fast-, moderately fast-, and slow-rising hypothetical storm surges that attained an elevation in Breton Sound of 10 feet (Figure 182) for the four cases described earlier (Figure 178). Bretschneider and Collins (1966) predicted that (1) changing the levee configuration and (2) including or excluding the MRGO channel would each affect surge transmission to the IHNC for all rates of surge development in the Lake Borgne estuary.

For the most rapidly rising surge, the IHNC does not rise at all without the MRGO (Cases I and IV), because there is insufficient time for the surge to propagate up the estuary. For the moderate rate of rise, the surge that reaches the IHNC without the

MRGO has a lower peak elevation and a narrower peak signifying a shorter duration of high water conditions. For the slow rising surge, the peak elevation in Lake Borgne is attained, or nearly attained, in the IHNC for all cases, after lags of varying duration, but the width of the peak is reduced without the MRGO (Cases I and IV). The period above 8 feet, or the 80 percent exceedence interval, in the slow-rise scenario drops from 3.5 to 2.0 hours for the condition with and without the MRGO, respectively, with the new levees in place (Cases III and IV). In all cases, the predicted effect of building the "proposed" levee system that now forms the throat of the funnel (Cases III and IV) is to hasten the onset of peak surge and to lengthen the period of highest water.

Given these results, it is interesting to look at the consultants' summation of the effects of the channel.

> "It is seen that the effect of the Mississippi River-Gulf Outlet is almost negligible for all large hurricanes accompanied by slow rising storm surges. It may be expected that once in a while a storm may occur which has a somewhat freakish, more rapidly rising surge in which case the Gulf Outlet may have a very marked effect. However, such a storm will not produce tides as high as the more critical hurricane tracks such as Betsy or the synthetic hurricanes."

The synthetic hurricanes referred to are those based on the 1959 SPH that were used as a basis for design of the GNO HPS. We now know that Bretschneider and Collins (1966) were generally correct with regard to peak surge height, if not duration. The surge of Betsy rose in 10 hours and Camille in 8. Katrina's surge rose in two stages that together lasted 25.5 hours (Figure 183). Nearly 17 hours of slow rise from 3 to 6 feet (NAVD88) occurred on August 28, the day before Katrina made landfall. A rapid rise of more than 8 feet occurred over the first 8 to 9 hours of August 29. Surge in the IHNC peaked at about 15 feet, an estimated 2 feet lower than the peak on the MRGO/GIWW at Paris Road (Figure 183). The lower peak stage in the IHNC was probably due largely to the flow lost over levees on the north and south sides of Reach 1 that had crown elevations varying from 12 to 14 feet. Drainage toward Lake Pontchartrain, which was 3 to 6 feet lower at the time, also lowered surge level in the northern reach of the IHNC.

All three storms that flooded New Orleans through the open 'back door' were accompanied by what Bretschneider and Collins (1966) would classify as slow rising surges. The study could have been interpreted as cautionary with respect to the

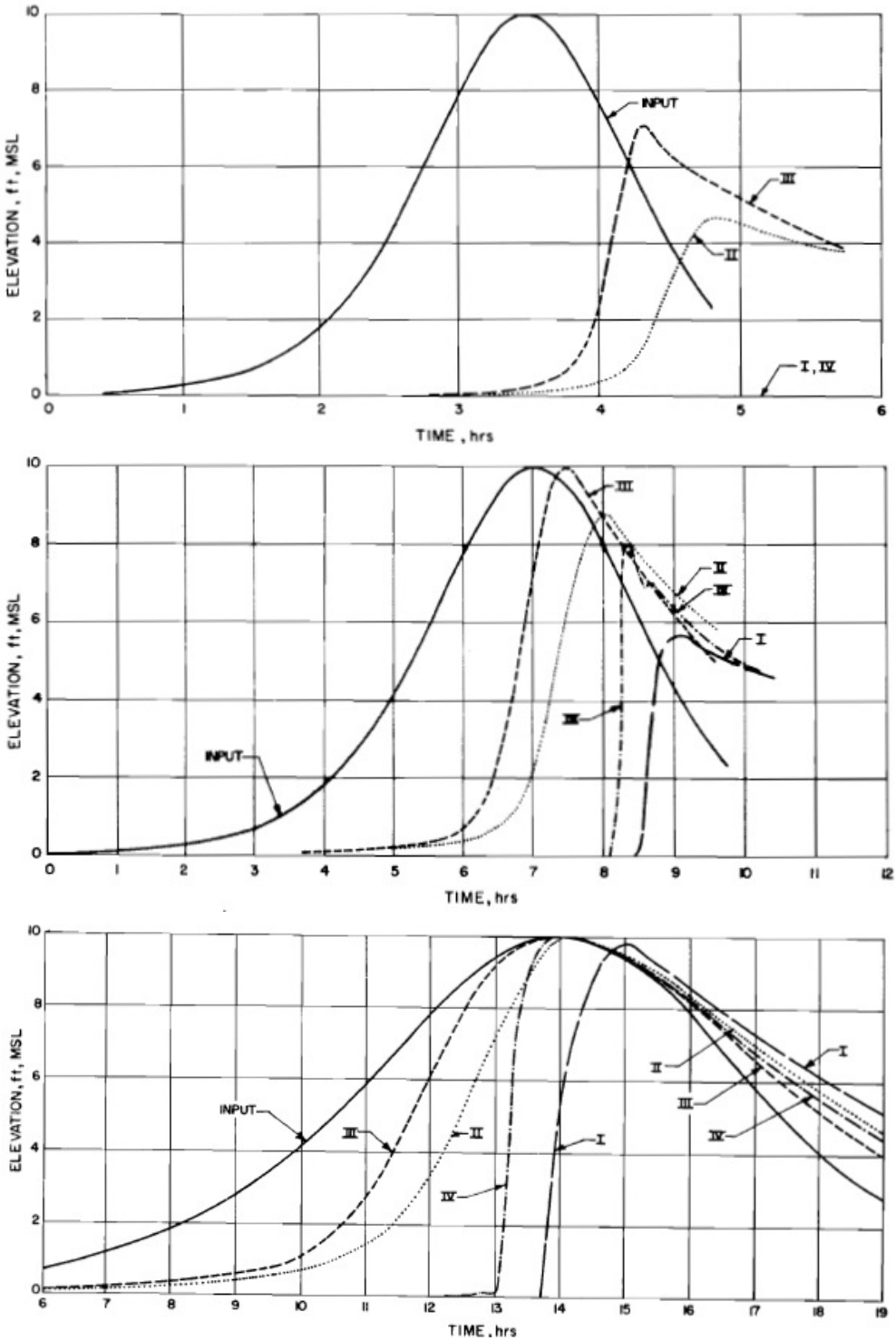


Figure 182. Simulated surge in the IHNC for a given input surge in Lake Borgne with a peak elevation of 10 feet (MSL). The top panel is for a rapidly rising surge that peaks in 3.5 hours. The middle panel is for the same surge that takes 7 hours to peak, while the bottom panel is for a surge that reaches a zenith after 14 hours. The numerals indicate the case tested: I - Existing levees without MRGO, II - Existing levees with MRGO, III - Proposed levees with MRGO, IV - Proposed levees without MRGO (Bretschneider and Collins 1966).

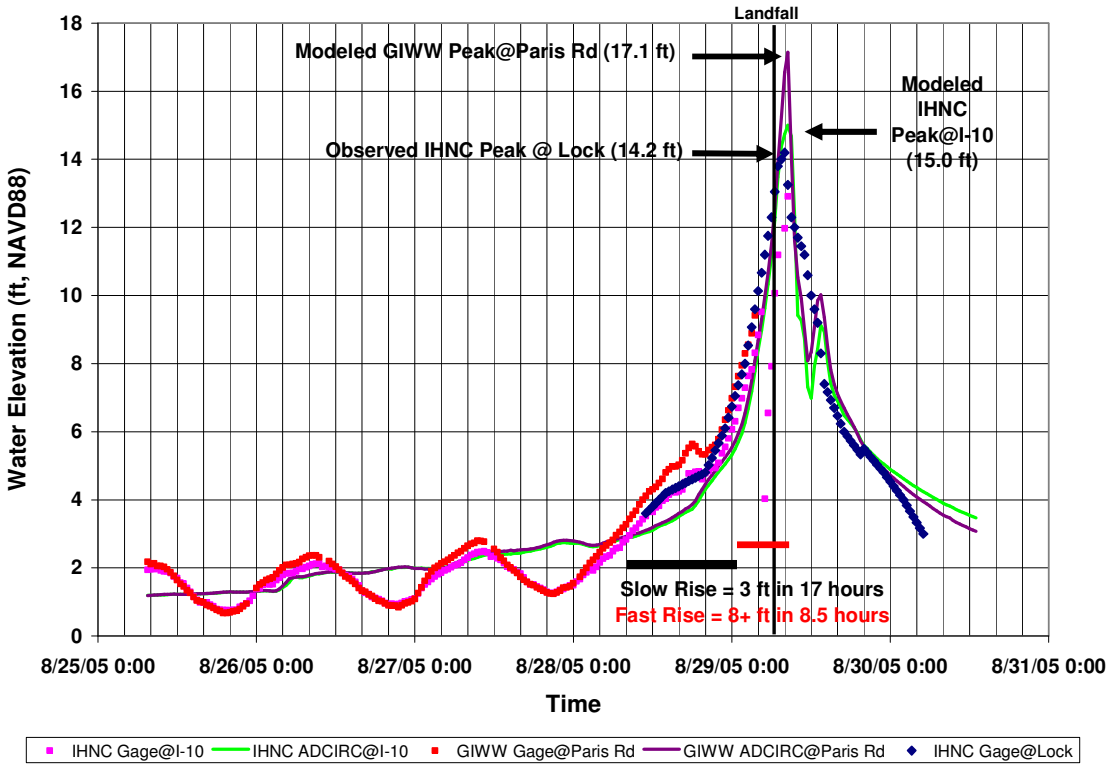


Figure 183. Hurricane Katrina surge in the IHNC and on the MRGO/GIWW at Paris Road. The only gage that recorded the peak was at the IHNC Lock (blue diamonds). The USGS gages on the IHNC at the I-10 bridge (purple squares) and on the MRGO/GIWW at Paris Road (red squares) both malfunctioned at about 10 feet, but all three gages recorded a slow initial rise of 3 feet over 17 hours, followed by the onset of a rapid rise over the first 8.5 hours of August 29. ADCIRC model results give the best estimate for peak elevations in the IHNC at the I-10 bridge (green line) and for MRGO/GIWW at Paris Road (purple line).

design of the hurricane protection system then being built. It showed that surge elevations experienced at the apex of the funnel near the Paris Road bridge would be transmitted to the IHNC without diminution, and that levee and floodwall crown elevations throughout this system would have to be essentially the same throughout. Further, it showed that the levee proposed along the south bank of MRGO Reach 2 would enhance the build-up and transmission of surge into the City. What is disturbing is how the Bretschneider and Collins (1966) study, rather than ringing alarm bells, appeared to lull the USACE back into complacency. The integration of a large, free-flowing ship channel into the GNO HPS clearly posed a great hazard to the City, yet never led to a proposal for the flood gate that IPET (2006b) has since found was so obviously needed.

> "The Reach 1 GIWW/MRGO section is very important in determining the magnitude of the storm surge that reaches the IHNC from Lake Borgne

> and Breton Sound. If the hydraulic connectivity between Lake Pontchartrain and Lake Borgne is eliminated at a point within this section of channel, tide or surge to the west of this point will become primarily influenced by conditions in Lake Pontchartrain" (IPET 2006b, IV-6-2).

Ironically, the gate that was proposed as part of the original HPS "Barrier Plan" at Seabrook, where the IHNC meets Lake Pontchartrain (Figure 158), would have increased flood level in the IHNC for any surge originating in the Lake Borgne funnel by restricting drainage toward the Lake. Hurricane Betsy provided a "wake-up call" that should have alerted USACE designers developing the GNO HPS that the original plan developed in the 1950s - prior to MRGO construction - was too exclusively oriented toward a Lake Pontchartrain surge. Hurricane Camille provided yet another demonstration of the way a Lake Borgne surge would outflank this protection scheme. Currently, designs are being developed to overcome the shortcomings of this "Maginot Line." The barriers originally proposed for two of the Lake Pontchartrain passes (Figure 158) are being augmented with a third to address surge arising in the funnel. Most concepts include a levee through Lake Borgne to break up the funnel geometry, with a gate on the GIWW and possibly a second to accommodate drainage and shallow-draft traffic on a deauthorized MRGO (Figure 184).

Investigations Following Hurricane Camille

Hurricane Camille was the second most powerful storm at landfall to strike the United States, and only one of three since 1851 that would now be classified as a Category 5 hurricane (CPI 26.84). The eye followed a north-northwest track that passed east of the mouth of the Mississippi River and only about 25 miles east of the path of Katrina at the latitude of New Orleans (Figure 185). New Orleans and its eastern suburbs were exposed to sustained winds in excess of 100 mph, 20 to 30 mph higher than during Katrina. Maximum winds of 200 mph were experienced in the vicinity of the landfall near Waveland, Mississippi. Camille was a compact storm compared to Katrina, however, with hurricane force winds extending less than 75 miles from the center of the storm, compared to more than 100 miles for Katrina. Camille passed the latitude of New Orleans with a forward speed of between 12 and 15 mph, about the same as Katrina. The most important difference in terms of damage caused and loss of life is that Camille generated a surge in excess of 20 feet over only about 20 miles of the Mississippi coast compared to nearly 60 miles for Katrina (Figure 186). The greatest damage in Louisiana occurred along the Mississippi River in Plaquemines Parish downstream of the GNO. As with Katrina, the surge exceeded the crown elevations of the mainline Mississippi River levees and poured water into and across the River from east to west.

The highest storm surge elevations documented in Louisiana were from the MRGO funnel (Figure 187), and were between 10 and 11 feet (NGVD29) from Shell Beach into the IHNC, though the peak level dropped to 6.5 feet at the Seabrook opening to Lake Pontchartrain (Figure 188). The Camille surge rose in the funnel over about 8 hours, compared to 10 hours for Betsy and 25 hours for Katrina. Although the surge attained essentially the same peak elevation in MRGO Reach 1 and the IHNC as it had during Betsy, flooding in the GNO was limited to a minor breach on the west bank of the IHNC near the lock, that allowed water into the Upper 9th Ward (Figure 168). The difference was attributed to local efforts to build up the 40 Arpent levee and to the new federal floodwall built on the IHNC adjacent to the Lower 9th Ward.

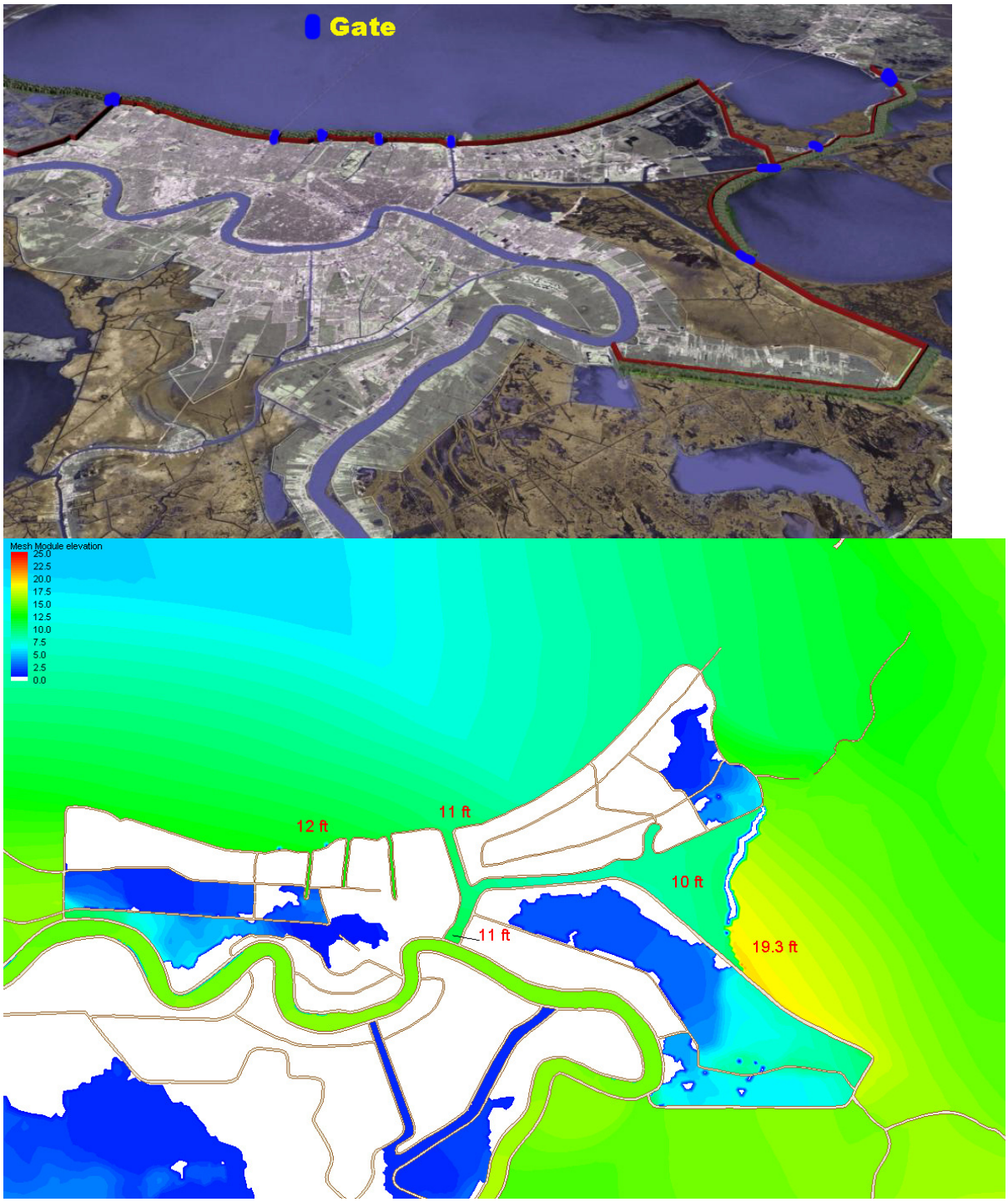


Figure 184. An improved hurricane protection design for closing New Orleans' "back door" (top). ADCIRC model output for Katrina (bottom) showing that the surge would be lowered in the IHNC and within the apex of the funnel but would be increased along a portion of the MRGO Chalmette Extension levee.

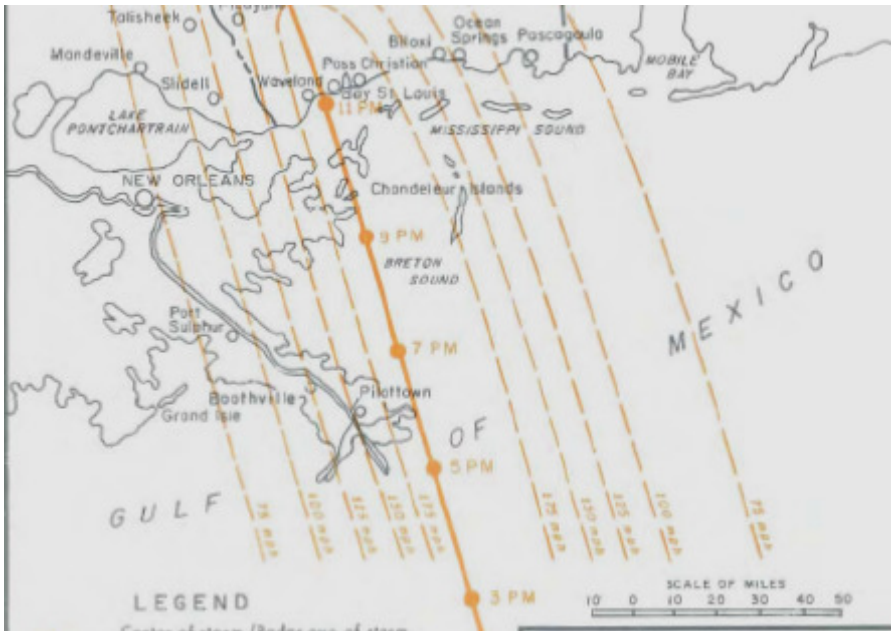


Figure 185. Path of Hurricane Camille and estimated wind speeds on the evening of August 17, 1969 (USACE 1970).

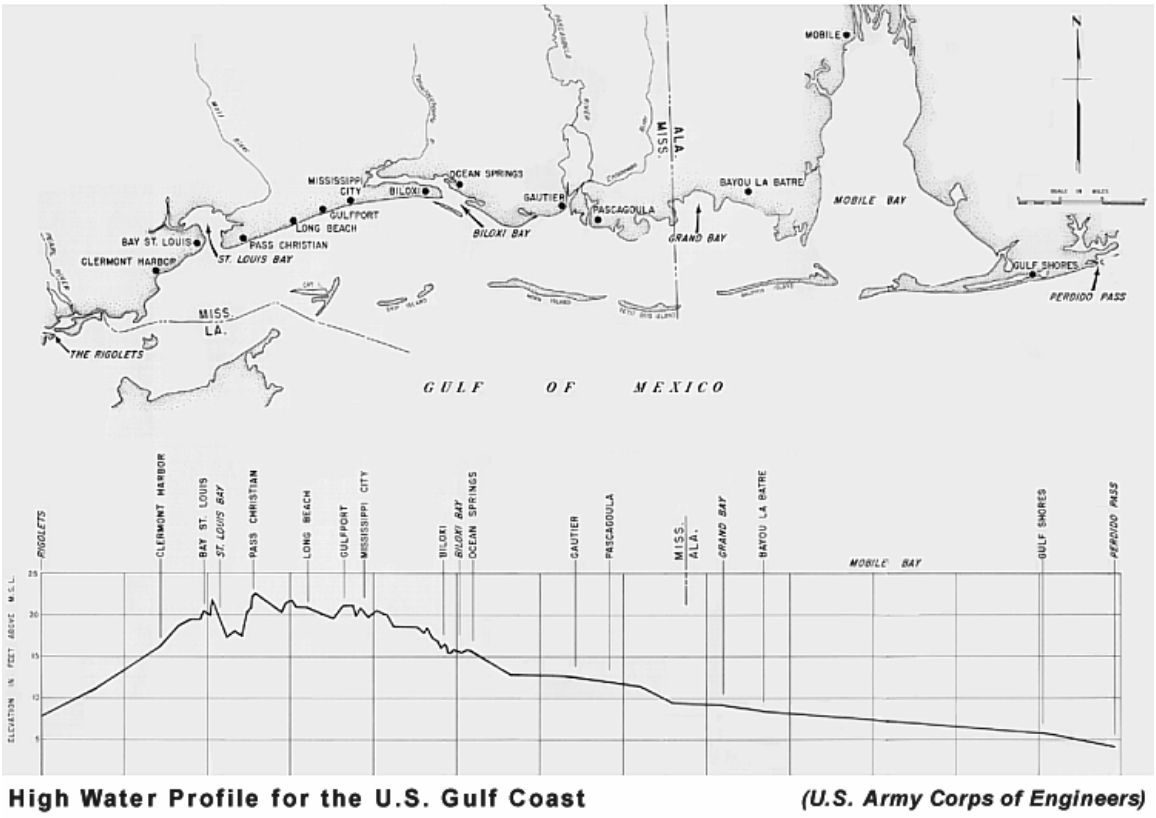


Figure 186. Maximum surge elevations on the Mississippi coast for Hurricane Camille.

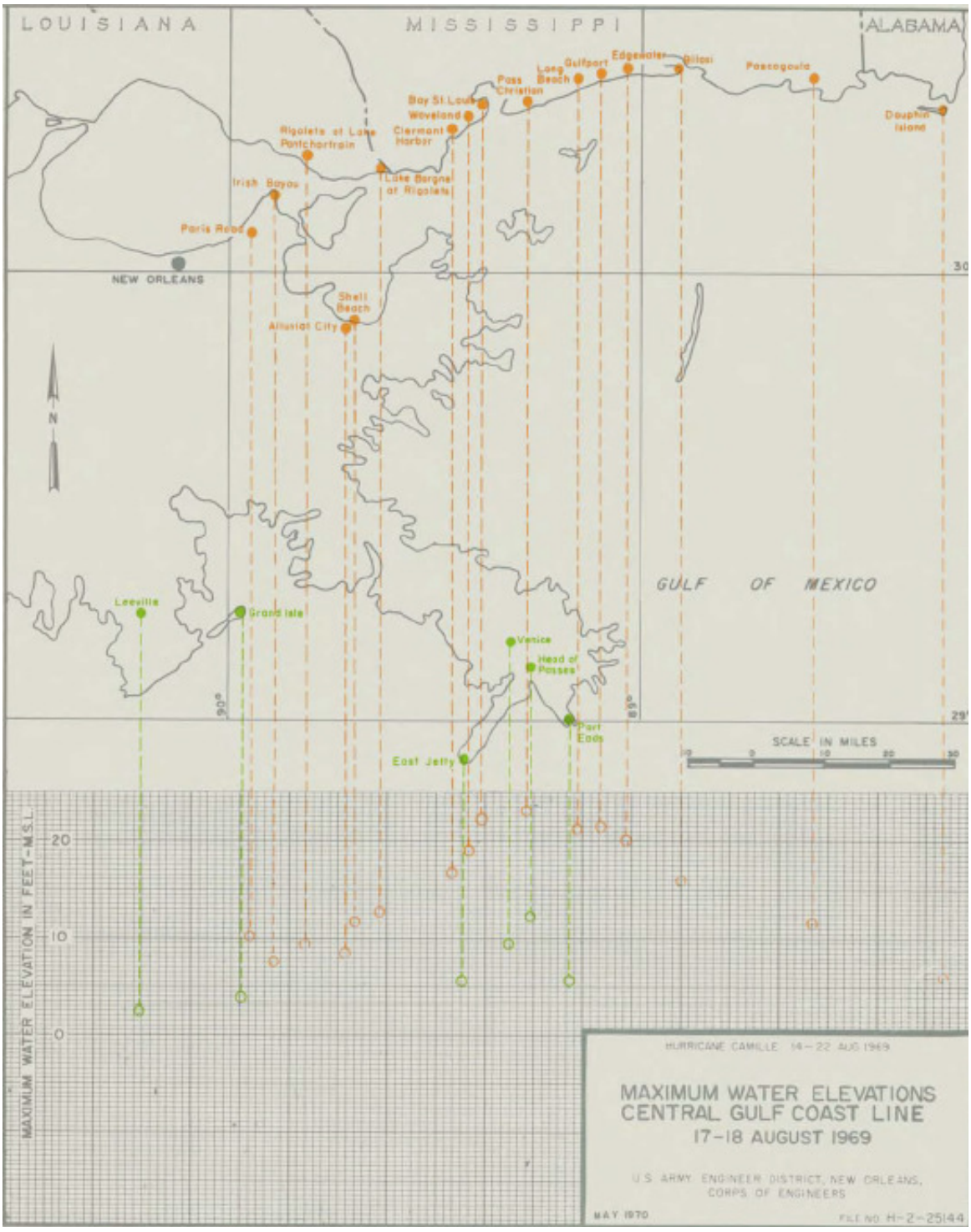


Figure 187. Maximum water levels recorded for Camille in Louisiana (10 to 13 feet) were in the MRGO funnel (USACE 1970).

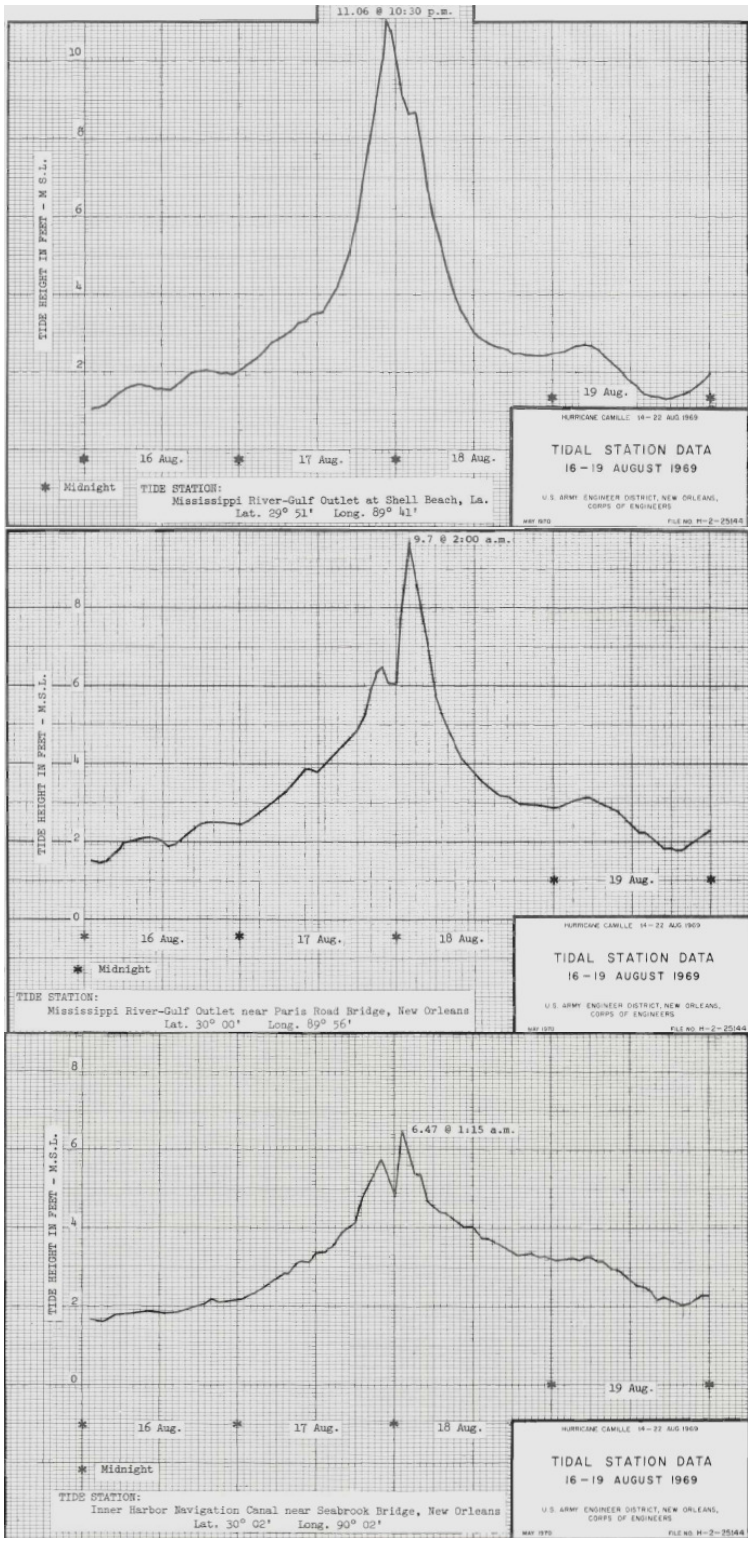


Figure 188. Camille surge hydrographs recorded at Shell Beach (top), Paris Road (middle), and at the mouth of the IHNC (USACE 1970).

Hurricane Camille was regarded as an outlier event with an almost infinitesimal return frequency by the National Weather Service (NOAA 1972). It had maximum wind speeds well beyond that of the maximum probable hurricane defined in the 1959 SPH analysis (Graham and Nunn 1959). The occurrence of Camille and a few other extremely intense Gulf hurricanes in the 1960s finally moved the National Weather Service to officially re-evaluate the whole basis for the SPH, as has been discussed previously.

Camille killed more than 260 people, primarily along the Mississippi Gulf coast and in inland areas, as far away as Virginia, affected by torrential rains. Only two persons died in Louisiana despite a surge in the Lake Borgne area that reached essentially the same height as that generated by Hurricane Betsy. Betsy had killed more than 80 people in Louisiana four years earlier. The GAO reported in 1976 that the USACE "estimated $91 million in damages were prevented during Hurricane Camille in 1969" by "the levee and floodwall work along the Inner Harbor Navigation Canal" (GAO 1976). It is not clear that Camille prompted any changes in the SPH concept or in the design of the remainder of the GNO HPS.

Investigations Following Hurricane Katrina

A consultant's report surfaced in late 2005 that purported to address the effect of the MRGO on storm surge (URS 2006). It was revised and repackaged in part as an appendix of the second report issued by the USACE IPET (IPET 2006a) and appeared again as an appendix in the final report (IPET 2006b, IV-6-1). IPET (2006b) cleared up some of the confusion left by the consultant's report but still concludes that the influence of the MRGO on regional surge maxima is small, generally less than a foot, for a range of storms including Katrina. These results are based on output generated by the ADCIRC hydrodynamic model also used by the LSU Hurricane Center (Mashriqui and Kemp 2006). The results are a useful contribution, but have been widely misinterpreted, as will be explained below.

It is important to understand that the consultant's report did not address the entirety of the MRGO channel, concerning itself only with the Reach 2 portion that extends through the marsh southeast of the more "critical" Reach 1 segment that has been discussed as the throat of the funnel. The consultant raised the ADCIRC computational nodes that previously were in Reach 2 of the MRGO channel to marsh elevation, to simulate a filling of that part of the channel. At the time that this work was done, a single bed friction value (Manning's 'n') was being used universally in ADCIRC to characterize channels, marsh, bays and the continental shelf. This was

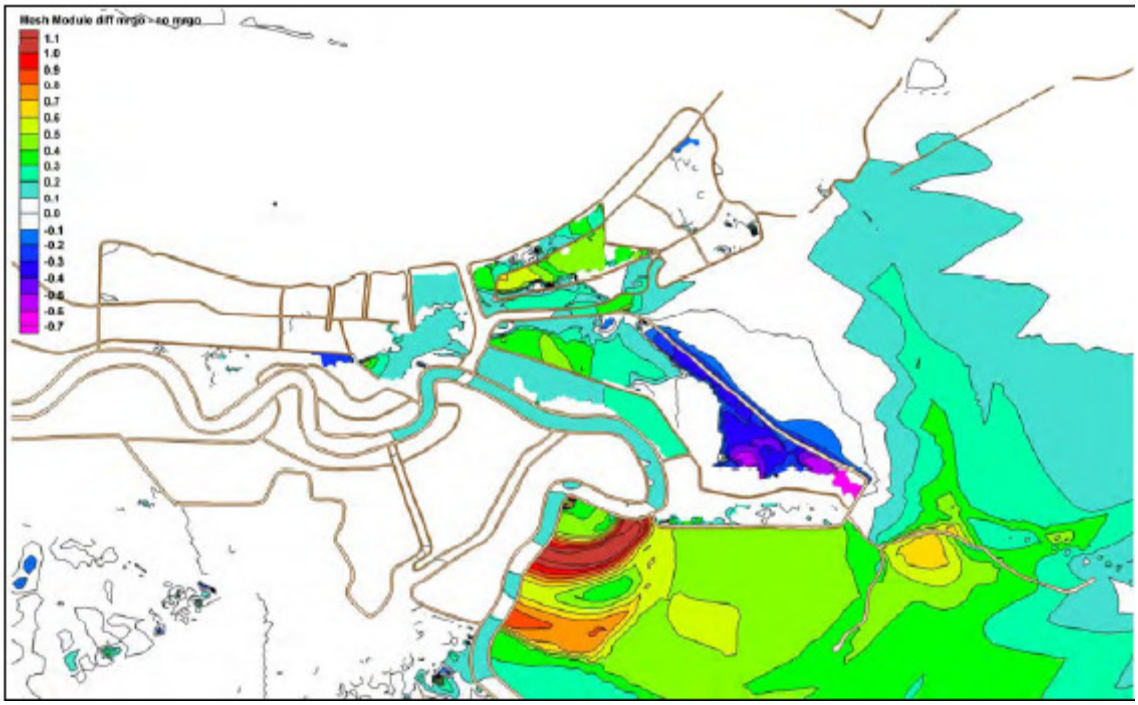


Figure 189. IPET (2006b, IV-6-34) depiction of the difference between the surge maxima surface developed by ADCIRC for Katrina with and without Reach 2 of the MRGO. Scale at left shows the differences ('With Reach 2' minus 'Without Reach 2') ranged from -0.7 to +1.1 feet.

reasonable for a regional surge forecasting model, but has been found to be inappropriate for more detailed work, such as apportioning flows between channels and marsh within estuaries, along the lines of what Bretschneider and Collins (1966) attempted four decades earlier.

Bretschneider and Collins (1966) used a Manning's value for the marsh of 0.08, compared to 0.02 for the channel, while a single value of 0.02 was applied everywhere in the ADCIRC version IPET used for assessment of the influence of MRGO Reach 2. So the channel bathymetry was eliminated **but the differential in friction between the channel and marsh was not addressed**. The consultants ran the model with and without MRGO Reach 2, extracted the maximum surge elevations attained during both runs, and then subtracted the "Without Reach 2" surface from the "With Reach 2" surface. They then created a graphic depicting the magnitude of difference throughout the area of interest (Figure 189). IPET (2006b) explains what they see as the very limited influence of MRGO Reach 2 in the vicinity of New Orleans for strong storm events as follows.

> "First, the MRGO does not influence the important preliminary east-west movement of water that drives the significant build up of surge in the

> early parts of the storm. Second, the northerly propagation of surge during the later stages of the storm are only minimally influenced by the MRGO because the increased hydraulic conveyance associated with the channel is very limited for large storms due to the large surge magnitude and especially due to the very large lateral extent of the high waters on the Mississippi-Alabama shelf that build up early on from the east. In addition, the propagation direction of this surge wave does not typically align with the MRGO and furthermore the southeasterly winds which align with the MRGO occur only very briefly" (IPET 2006b, IV-6-6).

Some of this rationale is reminiscent of the general conclusions of Bretschneider and Collins (1966), while other aspects are unique to the dynamics of Katrina. The surge generated by Katrina, in contrast to that of Betsy, reached a maximum along the south side of the funnel rather than in the throat. But it is worth taking a closer look at what the consultants and the IPET surge modelers actually found (Figure 189). First, it is apparent that while the difference in maximum surge elevation anywhere due to MRGO Reach 2 is small (-0.7 to +1.1 feet), the presence of this channel segment has effects that are distributed over a large area. Effects on surge maxima are generally greater outside the funnel than within it, suggesting that the main effect of the channel is to convey water away from the funnel, both northwest into Reach 1 and southeast beyond the eastern end of the Chalmette hurricane protection levee. An elongate blue area is shown on the south side of the funnel, indicating a reduction in surge elevation of up to -0.5 feet. This blue zone corresponds to two key features. The first is the Katrina surge maximum that stacked up on this portion of the levee along the south bank of MRGO Reach 2 (Figure 173). The second is the MRGO Reach 2 channel itself, that was eliminated in the 'Without' run.

Modeled flow direction vectors for the MRGO Reach 2 channel when the Katrina surge was highest show flow away from the zone of highest surge at both the northwestern and southeastern ends of this feature (Figure 173). Mean velocities in the channel at both ends of the high surge zone are in excess of 5 feet per second, and are much lower elsewhere outside the channel (Figure 172). A comparison of the ADCIRC generated mean velocity in the MRGO Reach 2 channel compared to that in the adjacent marsh at the time of peak surge, 6.2 and 3.0 fps, respectively, shows the difference due to depth alone (Figure 172). The flow direction is towards the entrance of MRGO Reach 1 at the apex of the funnel in both cases, but the mean depth of the MRGO at this stage is more than 50 feet, while the depth over the marsh is about 15 feet. So the channel acted to distribute water stacked up against the levee, thereby lowering the peak surge in the immediate vicinity of the Reach 2 channel segment. Flow out of the southeast end moved around the Chalmette levee and contributed to

increased elevations over a considerable area, particularly in the Caernarvon area where the increase was more than 1 foot. Similarly, water evacuated by the channel to the northwest raised the surge elevation in the throat of the funnel in the vicinity of the Paris Road bridge by about +0.5 feet.

Perhaps the most striking feature of the IPET Reach 2 analysis is, however, the effect of the presence of the ship channel on levee overtopping and flooding of developed areas (Figure 190). As is the case for all ADCIRC simulations including the 'with' and 'without' Reach 2 runs, levees and floodwalls are overtopped when the surge rises above the crown elevation. The model does not allow breaching, or accommodate rainfall or pumping. So, water enters the protected areas only for as long as the flood side water level is high enough to flow over the design crown level of the protection structure, and overtopping ceases when it drops below this level. Water accumulates inside the protected area and ponds in low spots.

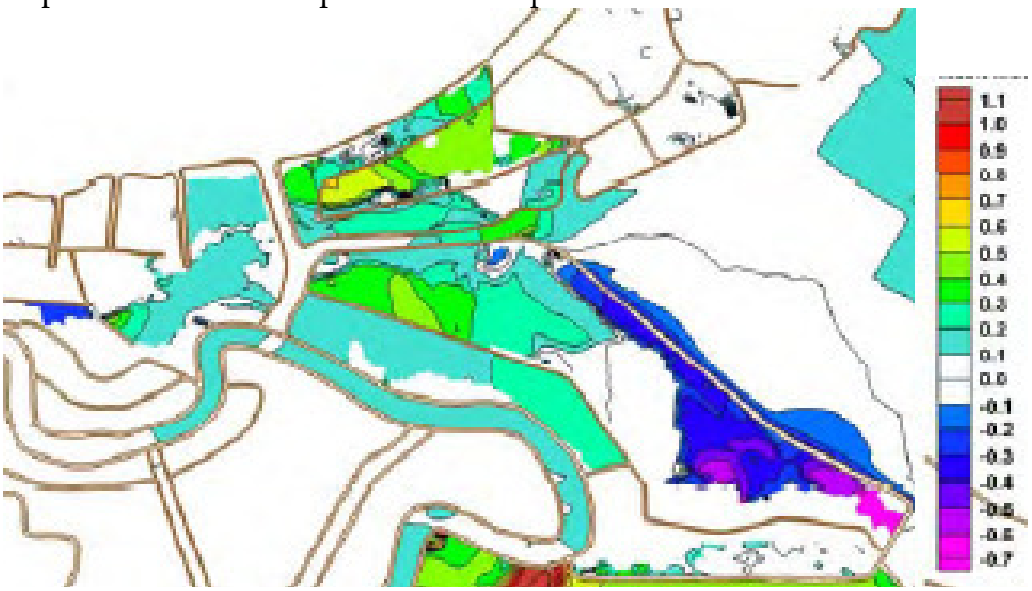


Figure 190. Close up of ADCIRC difference map showing the effects of MRGO Reach 2 on flooding of protected areas. Earlier arrival of surge delivered by MRGO Reach 2 caused overtopping to start earlier and resulted in increased flood elevations in protected areas in New Orleans East, St. Bernard and Orleans Metro.

The difference in maximum surge elevation predicted by ADCIRC on the flood side for the 'with' and 'without' Reach 2 simulations is small, and in the case of the IHNC, less than 0.1 foot (Figure 190). Yet the Orleans Metro protected area on the west bank of the IHNC is depicted with extensive areas of light blue and green indicating an increase in floodwater elevations of 0.1 to 0.4 feet due to the presence of MRGO Reach 2. More extensive areas of up to 0.6 feet of increased flooding are apparent in the New Orleans East protected area. Lesser increases are shown for the Lower 9th Ward and the Chalmette area of St. Bernard. Increased flooding of up to 0.5 feet is shown in the

northwestern half of the St. Bernard wetlands adjacent to MRGO Reach 1, while there is some lowering to the southeast caused by reduced overtopping of the federal levee in the surge peak zone (Figure 190).

The IPET investigators were so intently focused on surge maxima that they appear to have completely missed another effect of MRGO Reach 2 that is important to flooding over the levees and floodwalls. Levee designers are interested only in the maximum water level expected on the flood side of the structure, but when these structures are overtopped or breached, other aspects of the surge become important. The only plausible explanation for the increases in flooding predicted by ADCIRC within the protected areas is that the presence of **MRGO Reach 2 increased the period of time that the surge exceeded levee and floodwall crowns.** This would occur if the efficient conveyance of the ship channel caused the surge to rise more rapidly in MRGO Reach 1 and the IHNC, so that overtopping began earlier and lasted longer, regardless of the stage ultimately attained. This could occur without any detectable increase in surge maxima.

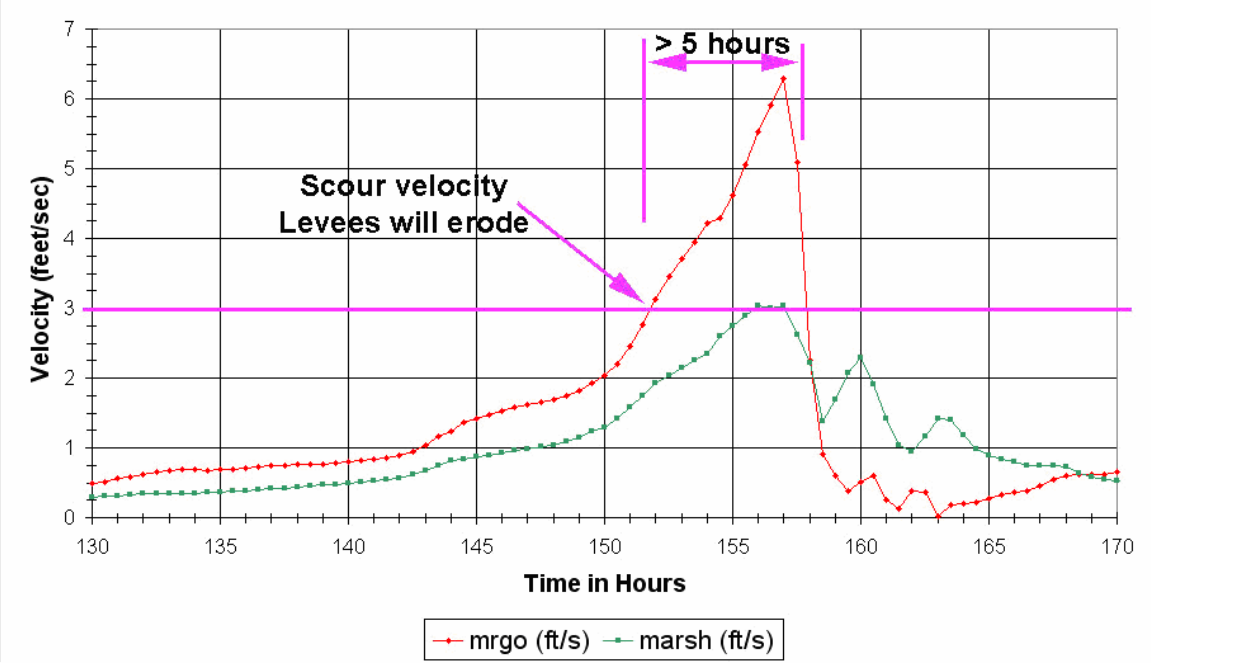


Figure 191 ADCIRC simulated flow velocity for hurricane Katrina from two points 2,000 feet apart in the vicinity of the Bayou Bienvenue control structure, one in the MRGO Reach 2 channel and the other in the adjacent marsh. The mean depth of the flow in the MRGO is greater than 50 feet, while that over the marsh is about 15 feet.

IPET (2006b, IV-6-4) notes that their study reached a similar conclusion to that of earlier studies going back to Bretschneider and Collins (1966) with regard to the effect

of the MRGO channel on relatively low storm surges, or, alternatively, during the early phase of a large storm surge.

> "In these situations, changes induced by MRGO Reach 2 are rather small, 0.5 feet or less, but this amounts to as much as 25% of the peak surge amplitude. When the long wave amplitude is very low, the surge is more limited to propagation via the channels. Once the surge amplitude increases to the point where the wetlands become inundated, this section of the MRGO plays a diminishing role in influencing the amplitude of storm surge that reaches the vicinity of metropolitan New Orleans" (IPET 2006b, IV-6-4).

The elevation of the surge in the IHNC at any point in the surge sequence is determined by a balance between flux entering through Reach 1 and that exiting to the Lake. The IPET analysis suggests that the first few feet of the surge rise at Paris Road and in the IHNC -- representing a volume of many millions of cubic feet of water -- enters initially through the channel. Later, as the surge rises in the funnel, more of the water enters from across the marsh. Given that mean velocity in the channel is at least twice that of the flow over the marsh (Figure 191), and that the depth of flow in the channel is three to four times that over the marsh, it is apparent that the 1,000 foot wide channel can deliver as much volume in a given period as flow across a 6,000 to 8,000 foot wide marsh cross-section. If the surge never rises high enough to cause flooding over levees and floodwalls, then the routing of the surge, whether over wetlands or through the channel, is of little interest, as the accumulated volume drains harmlessly out the north end of the IHNC into the Lake.

During the early stages of the surge rise, when the contribution of the MRGO Reach 2 channel is proportionally more important, flux into the IHNC is not limited by the dimensions of Reach 1. Later, as surge continues to rise in the funnel, and the relative importance of the MRGO Reach 2 channel contribution is diminished, flux into the IHNC through Reach 1 becomes limited by the conveyance potential of that connecting channel, and any additional surge buildup at the apex of the funnel has little effect on discharge to the IHNC.

The "critical" condition, when Reach 1 conveyance has reached its full potential, is indicated at least qualitatively in ADCIRC simulations by the development of a return flow in the GIWW that conveys rejected surge volume away from the entrance to Reach 1 (Figure 173). All water that enters MRGO Reach 1 contributes to the surge elevation along that connection. Some of this water at later stages in the surge was lost

as it overtopped the parallel levee system that confines Reach 1, again limiting discharge into the IHNC.

The IPET focus on late stage surge development appears to suggest that the volume introduced early when the flux was not limited by the dimensions of Reach 1, and when the contribution of the Reach 2 channel was proportionally more important, is somehow less consequential to the flooding dynamics than the volume introduced later in the storm sequence. Late stage surge in MRGO Reach 1 and the IHNC added to a surge volume already present. Another way to think about this is to attribute the base of the fully developed surge in Reach 1 and the IHNC to volume introduced early through the MRGO Reach 2 channel. The top of the surge peak is added later by flow across the marsh. Both sources contribute to the maximum elevation attained by the surge, which is limited by the conveyance of Reach 1. Because the water from MRGO Reach 2 gets to the critical reach first, however, the surge maximum is attained earlier than if all water came more slowly across the marsh, and flooding over levees and floodwalls in Reach 1 and the IHNC is also initiated earlier.

All of the ADCIRC runs discussed above applied a single bed friction factor everywhere. In a separate analysis, IPET (2006b, IV-5-94) investigated the effects of applying a spatially variable bed friction factor to surge simulations for Katrina. They found that merely increasing the Manning's 'n' value over marsh areas from 0.02 to 0.035 had significant local effects on the shape of the surge maximum envelope (Figure 192). Unfortunately, this analysis was not sufficiently detailed to allow for differentiation between the MRGO channel and the surrounding marsh. IPET (2006b) states that "the extensive network of connecting channels that occur throughout the marshes, cyprus forests and other regions in Southern Louisiana were not considered in these simulations."

Generally, applying a spatially varying bottom friction factor tended to increase surge height in open water and reduce it in wetland areas. This change reduced the predicted peak surge in the Caernarvon area south of the St. Bernard levee system by up to 2 feet (Figure 192). This estuary is not penetrated by large, deep, dredged channels. On the other hand, applying a greater friction factor for the marsh in the funnel was found to only slightly offset the higher levels generated in the open water to the north. It is possible that much of the surge reduction expected from the remnant remaining funnel marshes was diminished by the presence of the ship channel.

The effect of wetlands on surge reduction is an active area of research at present, in part spurred by the availability of high-resolution surge models like ADCIRC that

make it possible to accurately trace surge flow-paths and look at varying frictional effects along the way. Hurricane Rita, also a Category 3 storm, struck the southwest coast of Louisiana 26 days after Katrina, as has been discussed. Rita pushed a storm surge in excess of 10 feet across 100 miles of coast fronted by a sparsely settled 20 mile wide band of coastal wetlands, while Katrina came ashore on a part of the deltaic plain near New Orleans where wetlands have been greatly reduced over the past century by natural subsidence and man's activities.

A reduction of 1 ft for each 2.7 miles of marsh fronting a levee or floodwall had been derived from analysis of high water marks in southwest Louisiana for a number of storms prior to 1963. Mashriqui et al. (2007 – in preparation) recently found, based on data also collected in southwest Louisiana after Hurricane Rita, that this value is acceptable for marshes distant from large dredged channels. In the future, when surge models are modified to incorporate appropriate wetland friction effects, it is likely that we will find that much of the expected reduction in the funnel was offset by the presence of the MRGO.

Erodibility of Levees Constructed of Hydraulically Dredged Materials

During the storm, in contrast to what has been depicted by all of the ADCIRC model simulations for Katrina, the federal Chalmette levee along the south bank of the MRGO breached relatively early in the storm sequence both where the crown elevation was close to the design level of protection (17.5 feet), and in extensive reaches where it was deficient by 4 to 5 feet (Figure 193). As has been discussed in earlier sections of this report, the zone of significant degradation spread over several miles as the water rose, eventually allowing a flux on the order of 2 million cubic feet per second (cfs) across the federal levee alignment into the marsh on the protected side, and ultimately into developed areas of St. Bernard. There can be little doubt that this flux reduced the volume transmitted through the channel both to the northwest into the funnel throat, and to the southeast around the end of the Chalmette HPS.

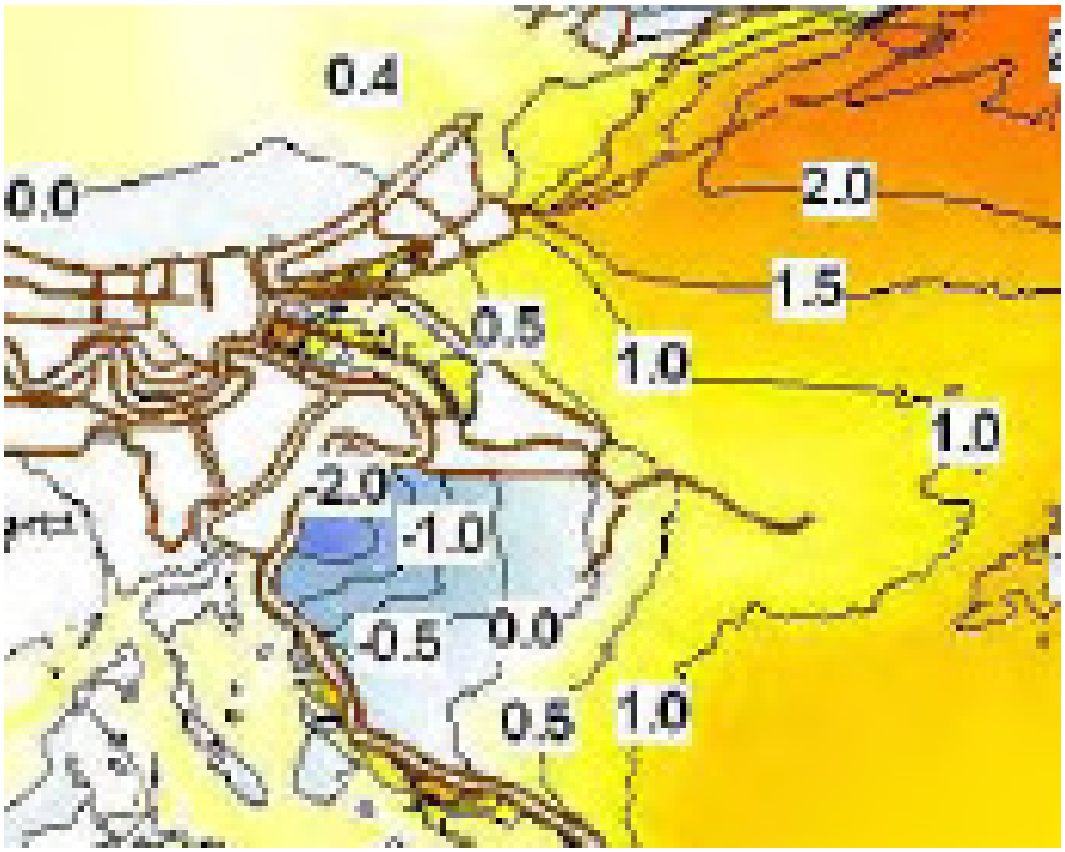


Figure 192. Difference in IPET ADCIRC surge maxima with spatially varying bed friction factor (0.035 for marsh and 0.02 for open water) and the base run (0.02 everywhere). Variable less base is shown in feet. Open water areas generally experienced higher surge (yellower), while surge in marsh areas without major channels like the Caernarvon bight south of the St. Bernard HPS was reduced by up to 2 feet (bluer). A similar reduction might be expected in the MRGO funnel but does not appear, possibly offset by the presence of the large channel (IPET 2006b, IV-5-105).

We have previously discussed the combined effect of wave and surge on the catastrophic washouts of the Chalmette levee and of the New Orleans East Back Levee where it parallels the GIWW at the eastern margin of the New Orleans East HPS (Figure 194). These two levee reaches share the characteristic of having been constructed by the mounding of material hydraulically dredged from the adjacent navigation channels (Figure 195). Team Louisiana investigators who visited these levees shortly after the storm were initially surprised by the volume of non-cohesive sand and shell materials exposed in breaches and washed out of these levees.

The idea of using hydraulically dredged fill to construct hurricane protection levees was considered unusual at the time, but was initially suggested by the mounds of dredged material that remained apparently stable at heights of 5 to 10 feet above ground level within the spoil retention area established during MRGO construction. This approach

Figure 193 Oblique view looking northeast across Chalmette levee toward MRGO channel between the Bayou Bienvenue and Bayou Dupre floodgates that was constructed of hydraulic fill from the channel (IPET 2006b, V-13-60). IPET notes that the pre-Katrina crown elevation of 19,000 feet of this levee reach was approximately 12 feet, rather than the 17.5 feet design level of protection.

Figure 194 View looking west along New Orleans East Back Levee along the north bank of the GIWW that was constructed of hydraulic fill from the channel in stages over a 3 year period (IPET 2006b, V-13-57). The GIWW channel is to the left in this photograph. IPET notes that 12,705 feet of this levee reach was completely degraded.

Figure 195 The New Orleans East Back Levee east of the Michaud Canal (purple line) was constructed using hydraulic fill dredged from the GIWW (IPET 2006b, V-77). The Chalmette and Chalmette Extension levees along the south bank of the MRGO east of the junction with the GIWW were constructed in the same way.

was later considered for construction of the Jefferson Parish lakefront levee, but was rejected. The method of construction proposed in GDM 3 (1966), and evidently used, was to dredge enough material from the adjacent channel to deposit a volume of dredged material between retention dikes sufficient to provide all necessary material for construction (Figure 196). After a period for dewatering and consolidation, material at the edges of the mound created would be moved to the centerline and shaped to provide a levee (Figure 197). This process would be repeated at intervals that allowed for settlement and consolidation, until the design grade of 17.5 feet was attained with a

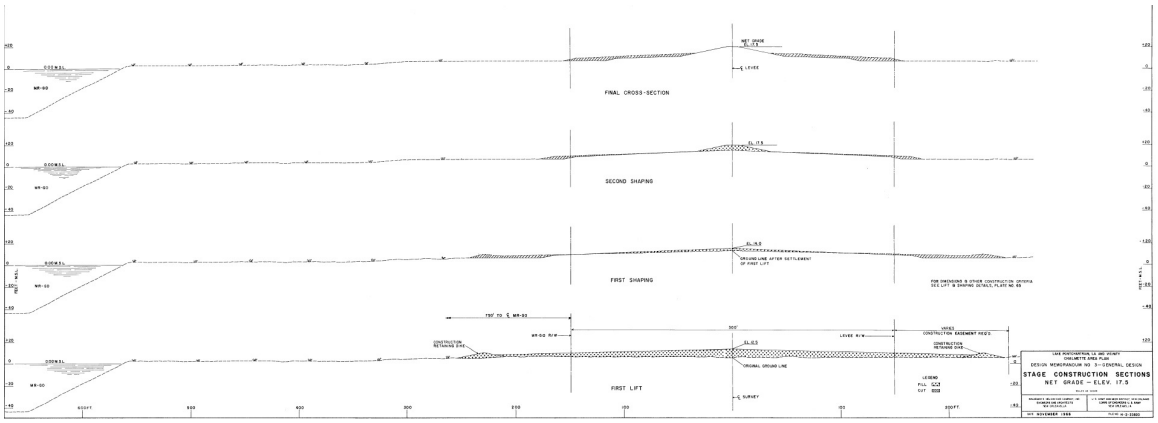


Figure 196 Construction of the Chalmette levee along the south bank of the MRGO using reshaped hydraulic fill (GDM 3 1966).

stable cross-section. These levees were built in the late 1960s and early 1970s, and were raised again in the mid-1980s. This methodology provided little opportunity for control of the materials that ultimately were incorporated in the levee, as those available were determined by what was available at the margins of the initial lift. The agitation caused by hydraulic dredging selectively retains the coarser-grained, heavier and less cohesive materials, while suspended clays that reached the disposal area would tend to be discharged over the retention levees or through drainage sluices. The result was that the sand, silt and shell materials contributed the bulk of the levee cross-section and much of the foundation.

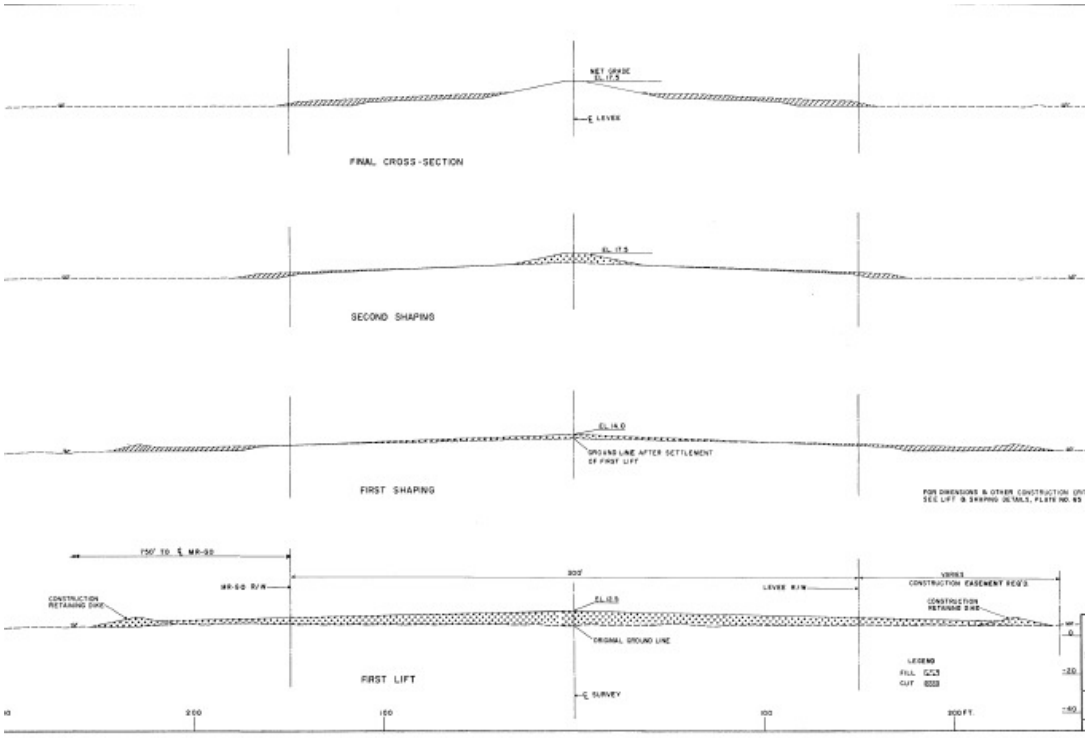


Figure 197 Detail of construction of the Chalmette levee using hydraulic fill dredged from the MRGO channel, showing the original fill lift and subsequent reshapings.

IPET (2006b, V-13-25; V-13-52) briefly summarized selected logs of borings made through hydraulic fill levees between 1991 and 2001. These logs were apparently found in files maintained by the New Orleans District that have not yet been made available to other investigators. Some logs showed the presence of a clay levee cap of variable thickness, while other borings showed non-cohesive materials at the surface.

The extent of degradation during Katrina of the levees built in this manner, coupled with the paucity of available as-built drawings and pre-storm boring logs has rendered it difficult to fully assess the contribution of the construction method and materials to the poor performance of these levees. Most investigators, including those affiliated

with the IPET, believe that it played a significant role. Fortunately, ILIT investigators sought to test this assumption by collecting shallow borings of surviving levee sections (Figure 198) and subjecting the materials obtained to erodibility testing in the laboratory (ILIT 2006, 10-22). ILIT employed the erosion index testing method developed by J-L. Briaud at Texas A & M University to measure the shear stress and velocity of flowing water required to initiate erosion of undisturbed, reconstituted and compacted levee soil samples.

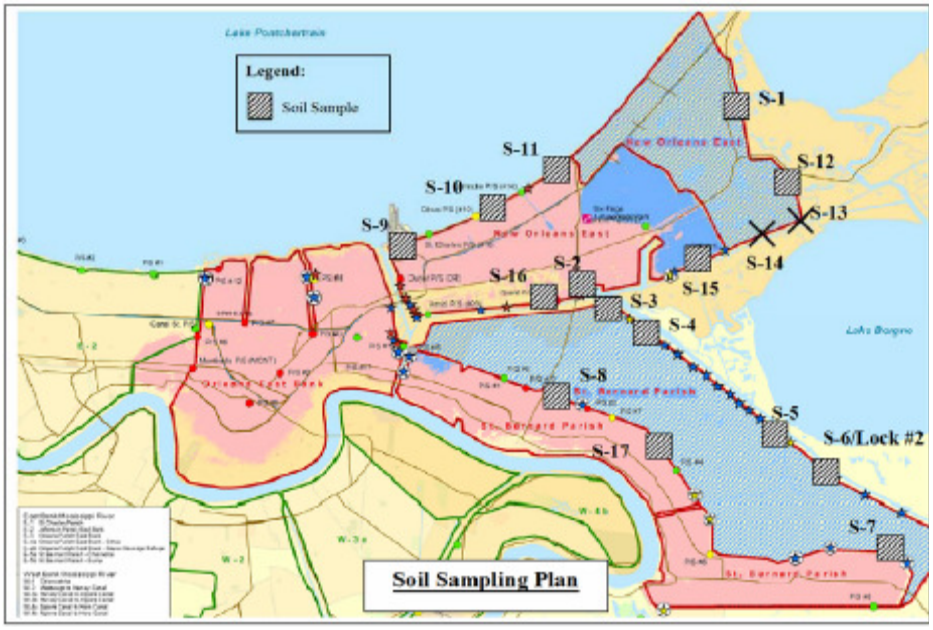


Figure 198 Locations of levee soil samples obtained from GNO levees by the ILIT for erodibility testing (ILIT 2006, 9-47).

Samples obtained from earth levees throughout the GNO HPS exhibited a wide range of erodibility indices from very low to very high categories (Figure 199), but those from the Chalmette (MRGO) and New Orleans East Back Levee (GIWW) tended to fall in the very high to high-erosion potential ranges. ILIT (2006, 9-52) found, furthermore, that MRGO samples that tested in the high-erodibility categories could be greatly improved with respect to erosion resistance by subjecting them to compaction in the laboratory (Figure 200).

The ILIT analysis showed a reasonable correspondence between levees that yielded high to very highly erodible samples and the prevalence of breaching. But Team Louisiana investigators hypothesized that another failure mode in addition to the shear applied by overtopping flows might also be a factor. It is well known that the non-cohesive silts and sands that made up much of the volume of the failed levees are highly susceptible to sudden loss of strength as a result of liquefaction. Hydraulically

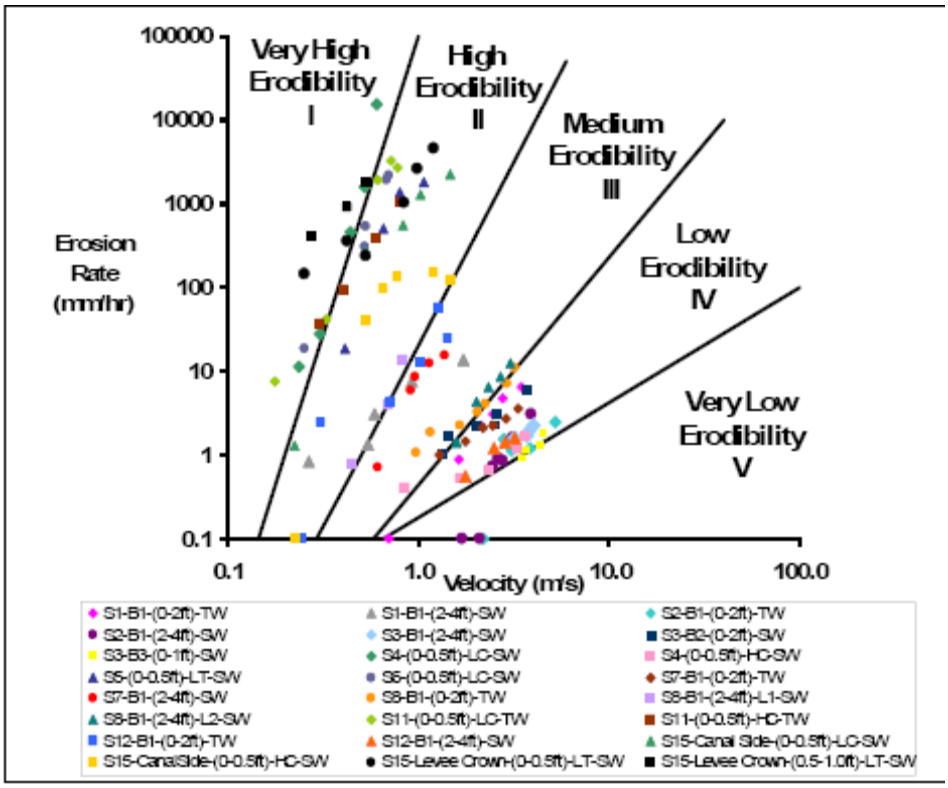


Figure 199. ILIT results of erodibility testing. Note results for samples from sites S4, S5 and S6 from the MRGO levee and S15 from the New Orleans East Back levee tend to plot in categories I and II, indicating 'very high' to 'high' erodibility (ILIT, 9-51).

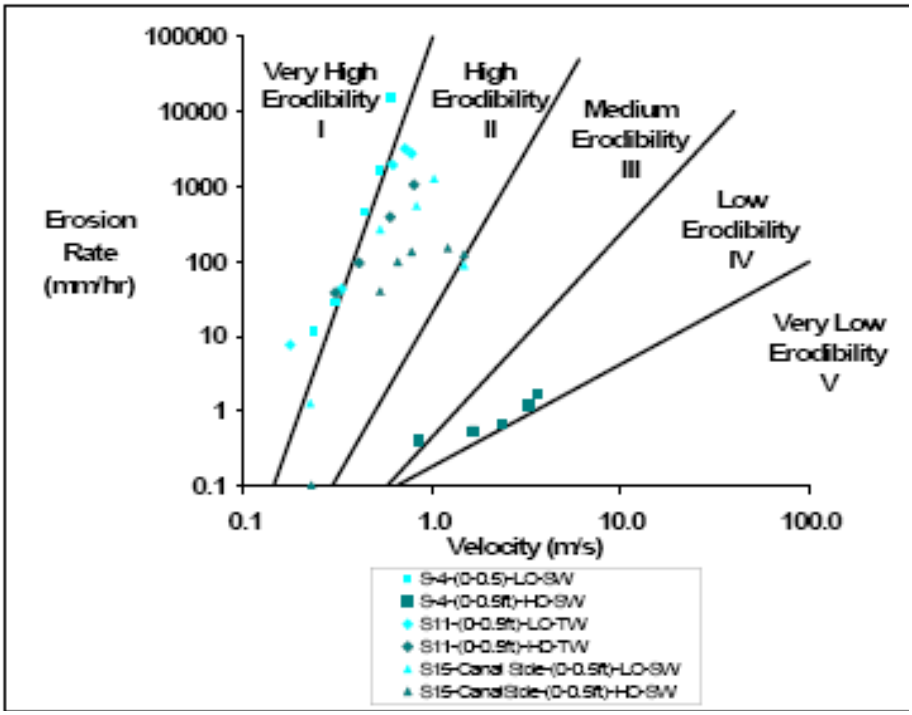


Figure 200. Effect of compaction on erodibility of MRGO (S4) levee samples (ILIT 2006, 9-52).

filled reclaimed lands are present in many coastal cities, and have proven particularly problematic during seismic events. This is the phenomenon of "quicksand," with effects dramatically portrayed in countless movies of people or animals sinking inexorably as they struggle just out of reach of would be rescuers. Liquefaction occurs when water saturated materials of this type are subjected to shaking during seismic events, or possibly as a result of periodic wave impacts. ILIT (2006, 10-3) also raised this possibility.

> "relatively loose materials (such as sands)…tend to be sensitive to vibrations and if they are over-excited (by cyclical perturbations such as earthquakes or heavy wave pounding), they will self-compact themselves and densify. During the self-compaction process, the soils temporarily become "fluidized" as the water between the soil particles rises to the surface…During the "fluidized" state, the foundation for the levee becomes unstabilized and can result in catastrophic failure…In seismic areas, levees are generally required to be designed against liquefaction failures, but the impact of wave perturbations on loose foundation materials is not well established."

ILIT goes on to state, however, that while "liquefaction may have been a partial contributor to some of the failures along the MRGO levee system," they did not "find strong evidence" for foundation failures due to liquefaction (ILIT 2006, 10-21). Team Louisiana investigators also did not observe clear evidence that liquefaction in the hydraulically filled levees contributed to foundation failures, but we suspect that it may have accelerated the rate at which breaches opened and widened through the levee crowns. While liquefaction causes loss of bearing strength on a flat surface, it results in high-density flows where a slope is present, as in a levee, and could provide a faster-acting failure mechanism than particle-by-particle removal. Given the extent of damage that the MRGO and GIWW levees sustained during Katrina, it would be surprising if it were possible to distinguish the effects of one failure mode over another.

IPET (2006b) does not mention liquefaction anywhere in the more than 6,000 pages detailing their investigations. IPET (2006b, V-13-63) does, however, note the presence of apparent back side slumping in at least one location along the MRGO levee that is difficult to explain merely as a result of surface erosion (Figure 201). But IPET (2006b) also has suggested that waves with a period as long as 15 seconds were part of the attack on levees surrounding the funnel. Cyclic forcing at that frequency is far more likely to penetrate to the interior of sand-filled levees than would be the case for locally generated waves with periods of 7 seconds or less. The state of destruction of the hydraulically filled levees after Katrina made it virtually impossible to establish any

complex combination of failure modes. It is possible that foundation failure induced by liquefaction played more of a role than has heretofore been suggested. This merits further investigation, as the levees along the MRGO and GIWW have been rebuilt on the same foundations that was present before the storm.

Figure 201 MRGO levee failure southeast of Bayou Dupre that resulted in removal of both the levee and its foundation. IPET (2006b, V-13-63) suggests that the depth of the scour may be indicative of a slope stability failure rather than simply erosion due to overtopping. It is also possible that such a back-side failure could indicate foundation weakening due to liquefaction and selective removal.

To Armor or Not to Armor, That is the Question

It was noted earlier that USACE levee designers generally dismissed the need to provide protective covering for the earthen levees constructed as part of the GNO HPS, beyond that afforded by establishment of a grassed surface. They did this typically in a single sentence repeated without change in each GDM. The following example is from the Chalmette levee GDM from 1966.

> "Due to the short duration of hurricane floods and the generally erosion resistant nature of the soil along this project, no erosion protection is considered necessary along the leveed portion of the project" (GDM3, 1966, p. 19).

Along most of the Lake Pontchartrain lakefront, more resistant hauled clay soils were, in fact, used to face the levees, and wave berms and sea walls were placed in front of the levee faces at critical locations. These levees were not overtopped except by limited wave overwash and generally performed well. The sea dikes constructed along the boundaries of the funnel were provided with rip-rap protection at the toe on the channel side to retard erosion of the channel into the levee base, and some were constructed with stability berms to offset the relatively low strength of the soils used, but no additional protection, usually referred to as "armoring," particularly on the back side, was added to address erosion caused by expected overtopping within the statistical range inherent in the definition of the "significant" wave. Again, as was universally true throughout the GNO HPS, the design approach adopted was extraordinarily "brittle," or accepting of the potential for catastrophic failure in the event of even relatively minor overtopping (ILIT 2006, F-14).

Recognition of the failure potential might have inspired the USACE to carefully attend to maintaining levee crown elevations at the design level, but this was clearly not the case. Levee crown elevations, particularly in hydraulic fill reaches more prone to settlement, were allowed to drop several feet below the design grade over many years, thereby compromising the overall integrity of the GNO HPS. This deterioration took place without advising populations put at additional risk. Effective planning, or failing that, just official notice of the hazard, would have raised the level of public alarm that is unfortunately key to ensuring appropriation of emergency funding or establishing a sensible ordering of public priorities. The potential for action by the public through their elected representatives is thwarted if engineers with superior knowledge of deficiencies are complacent or silenced by dysfunctional bureaucracies.

Engineers involved in the design, construction and operation of the Dutch flood defense system remain mystified by the way the USACE continues to separate coastal levee design from the need to defend such structures from waves. The MRGO levee, for example, has been rebuilt to the pre-Katrina design grade, and the materials used for the patches are far better than those originally used. But the levee was not initially designed as a true coastal defense structure that could survive both surge and waves. The need for armoring has been treated as an add-on that must be separately justified, rather than as integral to the overall design. In the wait-and-see world of post-Katrina funding priorities, no assurance exists at present that the MRGO and GIWW levees will ever be upgraded to survive overtopping, despite awareness at the highest levels in the USACE that they remain among the most fragile parts of the GNO HPS.

The levees along the MRGO and GIWW were rebuilt over a 3 to 4 month period to provide emergency protection prior to the onset of the 2006 hurricane season. The

contractors were favored with a long stretch of good weather but still accomplished amazing feats just to get enough material into position in such a short period. Much of this material had to be barged in from Mississippi. The levees are very young, however, and will need continued attention for several more years to assure stability and to reduce erosion by rainfall as they remain largely unvegetated going into the first winter

Figure 202. Flood side of rebuilt Chalmette levee southeast of Bayou Bienvenue on the MRGO a year after Katrina. The yellow color is typical of Pleistocene clay imported by barge from Mississippi. Top panel shows view from the MRGO and bottom panel shows close-up of erosional rill formation in clay of levee face following late summer rains.

Figure 203. Stability berm on flood side of rebuilt MRGO levee southeast of Bayou Dupre a year after it was destroyed by Katrina waves and surge.

Figure 204. Armoring of transition using geotextiles, rocks and cement grout on east side of Bayou Dupre control structure viewed from the levee crown looking west (left panel), and from the protected side (right panel) a year after Katrina.

(Figure 202, 203). The only armoring in place has been constructed to strengthen the transitions between the levee and the control structures at Bayous Bienvenue and Bayou Dupre that washed out during Katrina (Figure 204).

CHAPTER 8

CONCLUSIONS

It is now over a year since Katrina struck New Orleans and Rita hit southwest Louisiana. The devastated northern Gulf Coast escaped a hurricane strike during the 2006 season allowing levee repair work to proceed without interruption. Louisiana State University (LSU) carried out its commission by the Louisiana Department of Transportation and Development (LDOTD) to assemble a team of Louisiana-based academic and private sector experts to "collect forensic data related to the failure of the levee systems around greater New Orleans." This group, later known as 'Team Louisiana,' was asked to focus on the hurricane protection system (HPS) designed and constructed over a 40-year period by the U.S. Army Corps of Engineers (USACE) for the East Bank of the Greater New Orleans area (GNO), including New Orleans East and St. Bernard Parish.

Team Louisiana had a year to distill findings made in the aftermath of the 2005 storms, and to carefully review the work that the Corps of Engineers IPET and external ILIT teams made available this summer. IPET, ILIT and Team Louisiana raced to gather information before evidence was destroyed in the rush to complete emergency repairs before the start of the next hurricane season. The efforts of the ILIT were truly courageous in many respects, and a gift both to the people of New Orleans and to the civil engineering profession. Although the National Academy of Sciences review board found that some findings of the USACE IPET were poorly supported, the quality of the underlying investigative work has not been questioned (NAS 2006). These teams focused primarily, though certainly not exclusively, on floodwalls known from early on to have failed without overtopping. Less critical scrutiny was directed toward other points where floodwaters entered the City either through or over protective structures with crowns below design levels. We have, accordingly, given more attention to the flooding where overtopping played a role, with or without breaching. This focus drove us to look in more detail at flooding associated with the Lake Borgne surge, at levee/floodwall crown height deficiencies, and the role of the federal navigation channels in flooding the City from the east.

Role of Local Governments, Levee Boards and the State

We have, in earlier chapters, alluded to interactions between the USACE and local cooperating or sponsoring agencies that affected the design of protective structures like the parallel floodwalls along the banks of the Orleans Metro drainage canals. Although the federal government had overall responsibility for the GNO HPS, the slow pace at which federal funds were made available ($3 to 5 million per year) led local agencies and their contractors to take a lead in many cases to get work started with local funds. As has been discussed, the USACE escalated the protection claimed for a completed Lake Pontchartrain and Vicinity Project from the 100-year to the 300-year storm level. This claim led local engineers to believe that designs originally proposed for some HPS elements were excessively conservative, and that an adequate system could be constructed more quickly and at lower cost without significantly sacrificing performance or reliability. In contrast, most investigators who have reviewed the designs after Katrina have concluded that factors of safety applied by the USACE were anything but conservative given the criticality of subsurface soils and the consequences of failure.

Levee districts are state commissioned entities advised on engineering issues by the LDOTD. LDOTD assumed this function after a reorganization in which it absorbed the duties of the earlier Louisiana Department of Public Works in 1978. The levee districts have the authority to raise funds within their boundaries for flood protection projects. They have typically been the cost-sharing sponsors for development of jointly funded federal hurricane protection systems. They also assumed limited responsibilities for maintenance of portions of the federal system that the USACE deemed substantially complete.

Another important factor came into play, however, after the National Flood Insurance Program (FIP) was established in 1968. Local governments sought to enhance economic growth by encouraging residential and commercial construction in new areas, often former wetlands, which were ringed by relatively low levees and subject to pumped drainage. With the advent of the FIP, development in these newly drained areas could proceed only if those who purchased properties there could also protect that investment with federal flood insurance. The Federal Emergency Management Agency (FEMA) would permit new areas to enter the FIP only if the levees and drainage system could be certified as providing protection against both the 100-year storm surge and 100-year rainfall event.

In the GNO, FEMA relied upon the USACE to provide engineering evaluations of flood risk in areas protected by both federal and non-federal levee systems. So local governments, rather than the levee districts, entered into discussions with the USACE to find out what minimal levee heights were necessary for certification against the 100-year surge event. If the USACE found that the perimeter levee system was high enough to prevent overtopping, then FEMA generally accepted this finding without requiring geotechnical or construction information normally called for to meet FEMA levee standards (44CFR 45.10).

Once the USACE determined that the levees were adequate or on the way to adequacy, the protected areas were then analyzed only for the capacity of the internal drainage system to remove rainfall. The probability that the perimeter levee would be breached or overtopped was not considered. The internal drainage capacity was then used to determine the Base Flood Elevation (BFE) that governed how high buildings had to be elevated, and the location of flood zones on the Flood Insurance Rate Maps (FIRMs). Development accelerated once BFEs and FIRMs were issued. Thousands of people moved into suburban areas on the outskirts of the GNO in some cases before levee and drainage systems were complete or fully functional.

Many supposedly safe homes flooded, primarily on the west bank, in the 1980s because the internal drainage system did not keep up with development, but also due to overtopping and breaching of levees caused by Hurricane Juan at the end of October in 1985. Juan was essentially a tropical storm when it affected Louisiana, but it raised coastal water levels south of the City as much as a major hurricane as it wandered just offshore for several days. Juan provided the same impetus for authorization of a federal West Bank and Vicinity Hurricane Protection System as the far stronger Betsy had for the Lake Pontchartrain and Vicinity Project twenty years earlier. The USACE described the 6-foot high embankments that failed during Juan as spoil banks rather than true engineered levees, even though they had previously determined them to be adequate for the FIP. After Juan, the levees were raised to a minimum grade of 7.5 feet, but it is possible that the requirement after Katrina for 100-year protection could be as much as 10 feet higher.

Local governments were driven by the need to enter the FIP to promote economic growth, and were primarily focused on internal drainage issues. The USACE saw FIP certification as a matter that was separate and distinct from its responsibilities for hurricane protection. The levee districts did not participate in the FIP discussions except after failures occurred. Given this history, the

multi-generational tension between the USACE and those being protected in the GNO is complex and easy to misunderstand.

The ILIT found that the GNO HPS failed to function as a system, but from a local perspective the HPS was successful for decades as a multi-purpose economic development tool that had an important role in facilitating drainage. Extreme rainfall events were far more frequent than hurricane surges, and apparently could be addressed, at least in the short-term, without a complete SPH-level hurricane protection system in place. Locals were all too ready to believe the glib assurances from the Nation's premier civil engineering organization that they were well protected against 100-year hurricane flooding.

Local elected officials saw their primary role as one of enhancing economic development and opportunity for their constituents, and regarded the need to enhance internal drainage efficiency as key. After extensive flooding of homes and businesses by rainfall in the GNO in 1995, Louisiana's Congressional delegation was effective in securing funding for a new USACE initiative, the Southeast Louisiana Urban Flood Control Project (SELA), with enactment of Section 108 of the Energy and Water Development Appropriations Act and Section 533 of the Water Resources Development Act (WRDA) of 1996. This project was to provide for flood control and improvements to rainfall drainage systems in Jefferson, Orleans, and St. Tammany Parishes. While funding to complete the perimeter HPS languished, construction began in 1997 on extensive improvements to pump stations and interior drainage canals in Orleans and Jefferson Parishes that could ultimately cost as much as $500 million. Information posted recently on the USACE, New Orleans District, website indicates that this work in the GNO is about 70 percent finished and should be completed in 2008.

Summary of Forensics Findings

We briefly described the unique geography and history that must be understood to appreciate the difficulties of protecting GNO from hurricanes. We then established storm and flooding chronologies for Katrina based on high water marks surveyed inside and outside the protection system after the storm. The ADCIRC hydrodynamic model provided a good tool to link these observations into an overall reconstruction of the surge and flooding sequences. Hurricane Katrina made landfall in Louisiana as a more powerful storm than the 1959 Standard Project Hurricane (SPH), but one that was better captured by the 1970s era NOAA SPH upgrades. The storm surge generated by Hurricane Katrina was similar to that anticipated by the 1950s SPH-based oceanographic analysis used

to set elevations for levees and floodwalls facing Lake Pontchartrain. On the other hand, the August 29 2005 surge exceeded 1950's era estimates made for the Lake Borgne funnel east of New Orleans, but was well predicted more than 24 hours before the landfall by the LSU Hurricane Center using the ADCIRC computer model developed over the past 15 years with funding from the USACE, amongst others (www.hurricanme.lsu.edu/floodprediction/).

We started the forensics analysis by showing how the design assumptions for the GNO HPS remained static despite growing scientific evidence that the threat posed by surge was actually far greater than originally estimated. This was followed by an examination of the design histories for individual HPS elements that showed a pervasive trend over time toward substitution of less reliable structures for more conservative designs. This was then broadened to a regional perspective, with an emphasis on effects of the surge generated in the Lake Borgne funnel. Here, we try to bring these threads together to identify critical problems with the management of the Lake Pontchartrain and Vicinity project that led, over time, to ever greater departures from the overall objective of the original authorization, to protect against the ***"the most severe meteorological conditions considered reasonably characteristic for that region."*** We read this objective as a mandate for continual reevaluation and adjustment that should have led to a project that evolved over time as more information became available.

In fact, the project did evolve as it grew to incorporate greater undeveloped areas to the east and west of the City and as the linchpin strategy of the original barrier plan was abandoned in favor of the "high-level" levees-only alternative. So, the original project was changed in fundamental ways without rigorous analysis of the trade-offs in level of protection afforded, maintainability and reliability. The 1984 Reevaluation, inspired by court decisions questioning environmental analyses supporting the design of the proposed Lake Pontchartrain barriers, could have been seen as an opportunity to critically examine the overall integrity and reliability of the system (USACE 1984). But the Reevaluation included no engineering analysis of the reliability costs of maintaining a long and intricate system of levees and floodwalls built on very poor soils.

The USACE concluded that it would cost more to shorten levees to leave out undeveloped wetlands than to continue the construction already started on the longer routes initially chosen to aid private efforts to drain and develop wetlands. A simple comparison of the initial construction costs expected to raise levees already built by private syndicates or local governments predictably was

determined to be less expensive than the cost of the barrier structures or of new shorter levee runs like the Maxent Canal alternative discussed below.

> "The two levee alternatives for the Citrus-New Orleans East area are essentially the same in terms of direct environmental impacts due to construction. Completing the existing levee system is a more economically feasible alternative than the Maxent Canal levee alternative, and is a more flexible alternative in that it protects, but does not preclude the future development of wetlands. Future policies and needs may be such that development of these wetlands is desirable. This additional planning flexibility also is a factor in favor of completing the existing levee system" (USACE 1984, p. 77).

In contrast to this history of expansion and modification, other decisions were made within the USACE organization presumably to control costs that were rationalized as a meticulous adherence to the largely discarded 1965 authorization. Contractors were required to use outdated benchmark elevations during construction, for example, while the erroneous substitution of the NGVD29 datum for the mean sea level datum specified in the original SPH oceanographic analysis – like that analysis itself -- never got a second look.

Question 1. Was the GNO HPS properly conceived to accomplish the 1965 Congressional mandate to protect against the "most severe combination of meteorological conditions reasonably expected?"

Answer 1. No. The initial meteorological and oceanographic analysis based on the 1959 U.S. Weather Bureau 1-in-100 year Standard Project Hurricane (SPH) was known to be obsolete by 1972, just as initial construction of parts of the GNO HPS was getting underway. The primary deficiency of the 1959 SPH was in the specification of maximum sustained wind speed, which the NWS had increased in 1972 by 20 percent, from 107 to 129 mph. The steady-state analytical approach used by the USACE to develop surge estimates was as sensitive to the effect of wind velocity as later numerical modeling approaches (i.e. SLOSH or ADCIRC), and should have alerted the USACE to the danger of underestimating wind speed. This analysis provided a design basis for setting the minimum heights above mean sea level for levee and floodwall crowns to resist overtopping by combined SPH waves and surge. We showed that a 20 percent underestimate of maximum winds could lead to a 40 percent underestimate of the predicted surge elevation using the standard wind tide

equation in use at the time the surge levels were set. In 1979 the NWS raised the maximum sustained SPH winds to 140 mph, a Category 4 hurricane!

The New Orleans District USACE was aware of this deficiency in the original analysis, as is indicted by testimony in the 1976 and 1982 GAO reports, but never revised the original SPH-based analysis to reflect the new understanding of threat even after being ordered to do so by the Chief of Engineers in 1981 (ER 1110-2-1453). New Orleans residents were not advised that the GNO HPS required significant improvements to meet 1-in-100 year SPH requirements, but, instead, the New Orleans District claimed at times that the GNO HPS would protect against a 1-in-200 to 1-in-300 year hurricane. No basis for this claim has been established. Numerous storms that passed close to the GNO area – before and after the 1965 HPS -- were more severe than the 1959 SPH.

The New Orleans District USACE missed opportunities to revise the original SPH-based analysis after the National Weather Service revised the SPH in 1972 and 1979, and when the SLOSH storm surge model came into use in 1979. SLOSH showed clearly that the GNO HPS, as it was constructed at the time, was vulnerable to overtopping by many possible Category 3 storms. This result was confirmed later by the ADCIRC model, as recently and publicly as during the FEMA 2004 Hurricane Pam Exercise.

(http://hurricane.lsu.edu/floodprediction/PAM_Exercise04/)

The USACE supported development of both surge models and was aware of GNO HPS vulnerabilities, but appeared to accept the inadequacy of the system with a complacency that undercut efforts to sound alarms and mobilize necessary upgrading.

Question 2. Were the levees and floodwalls at or above crown elevations specified, in designs for HPS elements, sufficient to resist overtopping by surge and waves associated with the Standard Project Hurricane?

Answer 2. No. Floodwall and levee crown elevations were built 1 to 2 feet low because of an erroneous assumption at USACE New Orleans District that an elevation of zero referenced to the National Vertical Geodetic Datum of 1929 (NGVD29) was equal to -- and interchangeable with -- local mean sea level (LMSL). LMSL was the relevant datum for superimposition of hurricane surge and wave height estimates from a 1950's era oceanographic analysis. In 1965, zero NGVD29 was between 1.3 and 1.6 feet below LMSL at different points in

the system, and floodwalls and levee crowns were constructed lower by this margin. This mistake was locked in for continuing HPS construction when the NOD adopted a policy in 1985, with the approval of the USACE Lower Mississippi Valley Division (LMVD), to explicitly use the outdated 1965 NGVD29 adjustment to control construction. As a result, no provision was made to account for the 3 to 4 feet per century subsidence rates characteristic of the GNO area even though this trend existed at the time of authorization. Crown elevation deficiencies ranging up to 6 feet at the time Katrina struck resulted in prolonged overtopping of floodwalls and levees along the IHNC and to the east in the Lake Borgne funnel that otherwise should not have overtopped or would have experienced overtopping for fewer hours. Prolonged overtopping led to catastrophic breaches into the Lower 9th Ward on the east and into Orleans Metro on the west, and contributed to the early failures of levees along the GIWW and MRGO. Early failure of the MRGO levee allowed the 32,000 acre St. Bernard wetland buffer to fill and overtop the 40 Arpent back levee while the surge was still rising in the MRGO, resulting in catastrophic flooding of the developed area of St. Bernard to an elevation of 11 ft (NAVD88).

Question 3. Did the USACE follow existing engineering practice and USACE guidance for construction of levees and floodwalls? Should issues about levee materials and floodwall designs have been detected and corrected by engineers equipped with the tools available at the time of construction?

Answer 3. No to the first question, and yes to the second. Weak soil strengths or potential for underseepage were evident in strata tested for the USACE during the early 1980s under Orleans metro drainage canal floodwall levees that failed. The potential consequences of these layers on levee stability were known to practicing engineers at the time but were missed or ignored because of inappropriate averaging of soil strengths on long levee reaches and across layers. Consolidation of soils beneath the I-wall levees on the 17th Street Canal should have increased soil strengths over time, but borings and soundings conducted since Katrina show that very soft clays in the failure zone still have strengths less than values assigned in 1981. Where Division-level reviewers identified potential problems, they were rebuffed by District personnel citing "professional judgment."

The New Orleans District USACE failed to conduct appropriate analyses of the potential for seepage to compromise levee and floodwall stability where shallow sand deposits occurred beneath the levee, such as at the London Avenue Canal. Design memoranda indicate reliance upon the Lane's Weighted

Average Creep method for underseepage analysis. This method was recognized in the profession at the time to be inappropriate for final design in a critical life-support structure. The presence of layered sands and clays should have led to analysis using more rigorous flow net and finite-element techniques in widespread use at the time, and specified in the governing USACE engineering manual for Design and Construction of Levees (EM 1110-2-1913, 1978 ed.). Sheetpile supported I-walls that were installed on levees with cross-sections too small to prevent underseepage also did not provide sufficient resistance when fully loaded, no matter what the sheetpile length. There is no evidence that rigorous analysis of uplift pressures was undertaken.

Idealized design templates were applied to long levee and floodwall reaches without adjustment for variable subsoil conditions or for variations in elevation on the protected side. IPET believes that such a mistake caused the levee supporting the I-wall levee to be built too low in the vicinity of the north breach into the Lower 9th Ward. The foundation failed early in the storm sequence at a water elevation well below the design level of protection because of inadequate resistance to overturning.

The New Orleans District USACE did not follow standard engineering practice or Corps guidance when evaluating whether to protect (armor) earthen sea dikes (levees) from erosion caused by waves in the funnel area east of the city. Such evaluations should have followed the 1954 TR-4 Shore Protection Planning and Design (Beach Erosion Board) or its successor, the Shore Protection Manual, first published in 1973. Instead of the required analysis, Design Memoranda for the New Orleans East and Chalmette Levees substitute the following disclaimer.

> "Due to the short duration of hurricane flood stages and the resistant nature of clayey soils, no erosion protection is considered necessary on the levee slopes."

These levees were not designed to withstand general overtopping, as was amply demonstrated in Katrina, but were expected to experience overtopping by waves greater than the significant wave provided in the oceanographic analysis. Many miles of the Chalmette and New Orleans East Levees were constructed of shell-rich sands with poor erosion resistance derived from the hydraulic excavation of the adjacent GIWW and MRGO channels, rather than the hauled clay soils specified for levees protecting urban areas (EM 1110-2-1913, 1978 ed.). Many of these problems could well have been detected if a

formal process existed for the state to review the designs and designs assumptions.

Question 4. Did the MRGO, a free-flowing, deep-draft navigation canal that pierces the HPS on its eastern side, compromise system performance?

Answer 4. Yes. The MRGO and GIWW channels provide efficient conduits to funnel surge into the heart of New Orleans. As a result, surge elevations peaked in Lake Borgne and the IHNC at nearly the same time, and at higher levels relative to levee and floodwall crowns, and earlier than would have been true prior to all of the wetland loss attributed to them. The effect of these federally constructed and operated channels on surge and waves has consistently been underestimated by the USACE from before Hurricane Betsy right through to the recent IPET report, as has the effect of accelerated wetland loss in the funnel area. One consequence of this institutional "blind spot" was that a hurricane barrier of the type proposed in the original pre-1980s HPS for the other two main passes into Lake Pontchartrain was never included for the MRGO.

The ILIT and IPET have indicated that the original "barrier" approach was a better design than the "high-level," levees-only HPS ultimately adopted twenty years after authorization. But our work indicates that disastrous flooding during Katrina from the Lake Borgne funnel and the IHNC would have been exacerbated by the barrier proposed at the Lake Pontchartrain terminus of the IHNC (Seabrook). On the other hand, the Lake Pontchartrain surge along the south shore might have been reduced by up to 5 ft by the barriers that were proposed for the other Lake Pontchartrain passes. This might have been enough to prevent one or more of the failures of the defective Orleans Metro drainage canal floodwalls built in the 1990s, and this would have greatly reduced the severity of prolonged flooding in Orleans Metro. Certainly, the barrier plan would also have reduced surge along the north shore of the Lake. Such trade-offs were never rigorously assessed when the decision was made to change the HPS design in such a major way in 1985 at a time when surge modeling techniques using SLOSH were available. Again, the level of protection was reduced without informing the population at risk.

Question 5. Was the system maintained and operated to assure the required level of protection through time? Specifically, how did the 40-year construction schedule impact system performance?

Answer 5. No. The GNO HPS was managed like a circa 1965 flood control museum. Design assumptions and policy made in 1965 continue to diminish the HPS today. Local sea level has risen 0.4 ft since the 1960s and much of New Orleans has sunk over 1.5 ft in the same period for a combined change of nearly 2 ft relative to sea level, but as IPET (II-78) noted,

> "It was not clear how projected subsidence rates were applied in structural elevation design, if at all. Subsidence was apparently not factored into the design freeboard allowance."

Prudent engineers operating in coastal Louisiana have made allowances for subsidence for a century. An analysis of all factors affecting levee elevation is required as part of FEMA Levee Elevation and Certification Requirements (44CFR65.10). It is inexcusable that this was not done for what was the most critical coastal protection project in the country.

Most public works structures would be scheduled for replacement or rehabilitation after 40 years, but planning for a more modern system was put off while the original project fell farther and farther behind. Because the USACE never completed the 1965 project, it could not legally pass responsibility for major maintenance or upgrades to the local sponsors, or initiate a new project to bring protection to a higher standard. Local sponsors kept levees and rights of way mowed, operated drainage structures, commented on USACE design memoranda, and participated in inspections. They were not, however, consulted on design or construction decisions. On the other hand, they were required to pay 30 percent of all costs incurred for a level of protection that appeared on some reaches to diminish over time. When Katrina struck, the crown height on most levee and floodwall reaches was between 1 and 3 ft low relative to current mean sea level, the only datum that is relevant to the oceanography of hurricane waves and surge. On the IHNC and MRGO, every foot of crown deficiency when the surge was above 11 ft meant that overtopping and levee erosion started a half hour earlier. A half-hour in the lifetime of a moving hurricane can mean the difference between success and failure.

Where Things Stand Today

The Lake Pontchartrain and Vicinity project employed generations of USACE employees and contractors at an estimated total cost of more than $500 million, while consuming nearly $200 million in state and locally generated funds. Prior to Katrina, it was estimated to be about 85 percent constructed, but was not

expected to reach completion until 2015. A similar West Bank and Vicinity project had been initiated on the other side of the river in 1986 after serious flooding associated with Hurricane Juan, as has been discussed. The West Bank HPS was expected to cost $330 million, again with a 35 percent local cost share, and was only 38 percent built prior to Katrina. Though it started much later, it was scheduled to be completed only a year after the east bank HPS. This much less capable system was not tested in 2005 to the same degree as the east bank HPS.

The USACE assured local sponsors over the years that the east and west bank HPS projects would, when completed, provide anywhere from 100- to 300- year storm protection for the entire GNO area. The 100-year estimate had at least some basis in the SPH return frequency analysis, as has been discussed. Escalation of USACE claims over time, without any corresponding increase in the elevation or robustness of protective structures, appears to have engendered an unfortunate complacency about the level of protection actually being provided.

The pre-Katrina total cost estimates for the two GNO hurricane protection systems, a total of about $1 billion, can be compared with costs recently compiled by the USACE for emergency repairs to the two projects since Katrina, and with estimates of additional expenditures necessary to achieve a more realistic 100-year level of protection (**Table 8.1**). Emergency repairs carried out by USACE contractors in the year since Katrina to return the GNO levees and floodwalls to the pre-storm condition have cost between $400 and $600 million, if the interim lakeshore drainage canal closures are included (**Table 8.1**).

The repaired HPS still provides a substantially lower level of protection than that originally authorized in 1965. Dr. Bob Bea, Co-Director of the University of California, Berkeley, Center for Catastrophic Risk Management, and an ILIT member, recently pointed out that "the repaired sections of the hurricane protection system are the strongest parts," but that "strong pieces embedded within weak pieces do not translate to a reliable system" (Bea 2006). Currently authorized projects to construct permanent lakeshore closures with pumps for Orleans Metro drainage canals will add an additional $100 to $200 million. Surge gates for the IHNC and MRGO are expected to cost at least $200 million more.

It is evidence of how pervasively under-built the system was that it has cost as much after Katrina to repair the GNO HPS to a marginally stable pre-storm condition as was spent in the previous 40 years. The USACE now estimates that

Table 8.1. USACE Estimates for GNO Emergency Repairs Completed and Added Investment Needed for 100 Year Protection ($millions)

Project	Low	High
Lake Pontchartrain &Vicinity		
Orleans Parish East Bank		
Emergency Repairs	250	350
Levees/Floodwalls/Armoring	505	1,095
Interim Orleans Canal Closures	65	170
Permanent Orleans Canal Closures	115	160
IHNC/MRGO Closures	200	200
Sub-Total Orleans East Bank	1,135	1,975
Added Cost for 100 Year Protection	*885*	*1,625*
St. Bernard Parish		
Emergency Repairs	80	100
Levees/Floodwalls/Armoring	210	340
Sub-Total St. Bernard Parish	290	440
Added Cost for 100 Year Protection	*210*	*340*
Jefferson Parish East Bank		
Emergency Repairs	30	50
Levees/Floodwalls/Armoring	180	350
Sub-Total Jefferson East Bank	210	400
Added Cost for 100 Year Protection	*180*	*350*
St. Charles Parish East Bank		
Emergency Repairs	10	20
Levees/Floodwalls/Armoring	40	190
Sub-Total St. Charles East Bank	50	210
Added Cost for 100 Year Protection	*40*	*190*
Summary Lake Pontchartrain & Vicinity		
Emergency Repairs	370	520
Added Cost for 100 Year Protection	1,315	2,505
Total Lake Pontchartrain & Vicinity	1,685	3,025
West Bank & Vicinity		
Summary West Bank & Vicinity		
Emergency Repairs	30	50
Added Cost for 100 Year Protection	445	1,350
Total West Bank & Vicinity	475	1,400
GNO Total Emergency Repairs	***400***	***570***
GNO Added Cost for 100 Year Protection	***1,760***	***3,855***
GNO Total Estimated Cost	***2,160***	***4,425***

between $2 and $4 billion will actually be required to achieve the minimal 100-year level of protection generally required to achieve Federal Emergency Management Agency (FEMA) certification for flood insurance purposes (Table 205). The level of protection that this will achieve should not be confused with the much higher Category 5 hurricane protection now being studied by the State and USACE. That will cost much, much more.

The Barrier Plan Revisited

Today, the USACE is responsible for the reconstitution of flood defenses for a ruined city that will offer a level of protection originally conceived in the late 1950s. Our discussion of the evolution of the SPH has shown that this level was known to be inadequate by the early 1970s. Since Katrina, the 1950s era barrier plan has been retrieved from mothballs, and is now being touted as central to virtually all restoration plans. Given this impetus, it is instructive to look at how the original plan would have affected performance during Katrina.

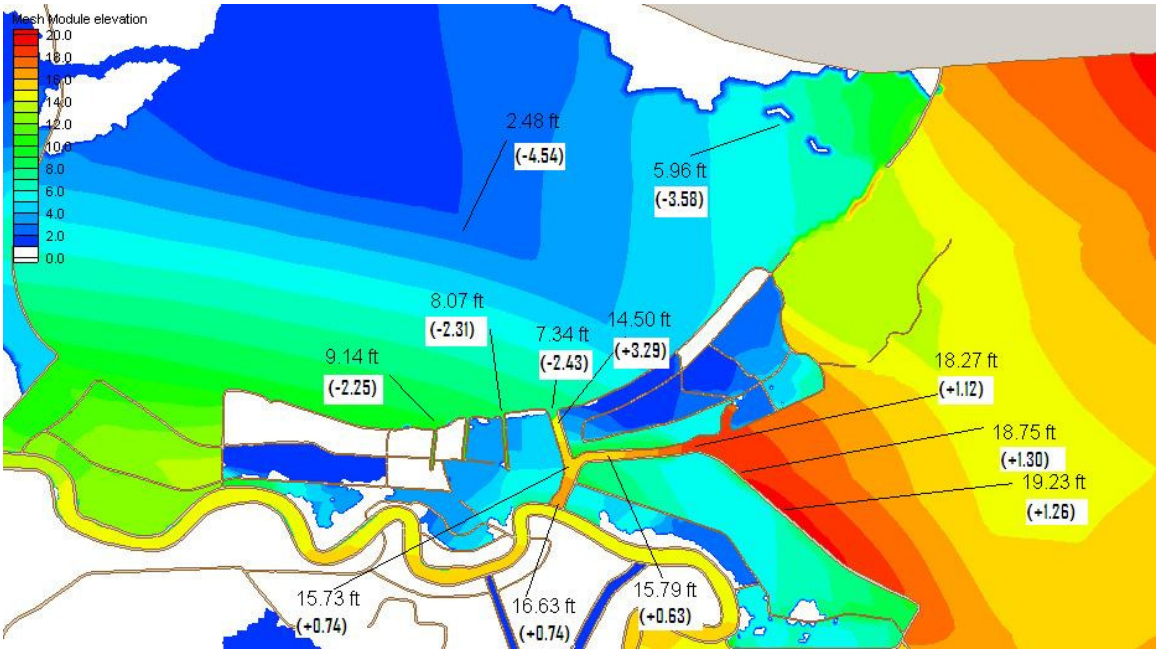


Figure 205 ADCIRC simulation of maximum elevations during Katrina with barriers originally planned at the Rigolets, Chef Menteur and Seabrook in place. Colors indicate maximum surge elevation according to scale at top left. Predicted Katrina maxima and the (+/-) deviation from 2005 conditions are shown for key locations.

The ADCIRC model can be modified to add the barriers at the three entrances to Lake Pontchartrain that were once planned. It can then be run to simulate the Katrina surge as if these structures had been constructed (Figure 205). If the gates were all closed while the storm was well offshore, the model run indicates

that mean water level in Lake Pontchartrain could have been maintained at just over +2 feet, rather than the + 6 foot mean level attained during Katrina, and the surge in the Slidell area along the north shore of the Lake could have been reduced by 3 to 4 feet.

Otherwise, the results do not show the large reductions in surge in Lake Pontchartrain that some have suggested. A substantial surge still would take place along the south shore, for example, as northerly winds tilted the Lake in that direction. The model predicts that the surge reduction along the south shore at the entrances to the Orleans Metro drainage canals would have been on the order of 2.2 to 2.4 feet, dropping the maximum level experienced by the lakefront levees and floodwalls from 10 to 12 feet to between 7 and 9 feet. If IPET is correct in its determination that the drainage canal floodwalls gave way when the lake level was between 6 and 9 feet, then the barrier structures might not have prevented these failures. On the other hand, they would have greatly reduced the volume and extent of flooding through the breaches over the next two days. Lake elevation along the south shore with the barriers in place would have dropped to its normal level within hours, rather than over more than two days.

The consequences of the proposed barriers would have being less positive for the St. Bernard and New Orleans East areas on the east side of the City that were affected primarily by surge rising in Lake Borgne that was conveyed west through the MRGO and GIWW funnel. The lake closure at Seabrook would have prevented drainage to Lake Pontchartrain and caused higher maximum surge elevations in the IHNC. The model predicts that the greatest increase, 3.3 feet, would have been observed just south of the Seabrook structure in the IHNC. The surge increase resulting from the Seabrook barrier would have diminished to less than 1 foot farther south in the IHNC, but would have contributed to substantial additional flooding and perhaps more levee breaks on this waterway. Increases in the surge maximum of 1.1 to 1.3 feet would have been observed along the MRGO from Paris Road to Bayou Dupre, as water that would have drained to Lake Pontchartrain was trapped in the funnel.

One of the flaws of the Lake Pontchartrain and Vicinity Project authorized in 1965 is inherent in its name. The primary threat to New Orleans at the time the oceanographic analysis was being conducted in the 1950s -- prior to the construction of the MRGO and before most of the suburban development in New Orleans East and St. Bernard -- was always seen as coming from the Lake. This is apparent in the planned Seabrook structure, which, had it been operated during Katrina, would have been built, like the French Maginot Line prior to World War

II, to address a threat coming from the wrong direction. The highest storm surges that have caused flooding in the GNO since Hurricane Betsy have come from Lake Borgne rather than Lake Pontchartrain, and this is likely to remain the most probable scenario going forward.

Looking Ahead

The integrated levees and barrier structures now being proposed to provide "Category 5" protection to the GNO are similar in many ways to the barrier plan proposed in the late 1950s and authorized after Hurricane Betsy. Clearly, given the model results above, improved levees and surge barrier structures can both be useful, and can be intentionally built to provide redundancy that will improve reliability. The same can be said about multiple levee lines separated by restored wetland areas designed for short-term storage of surge waters. It appears, however, that most of the proposals being presented continue to rely on legacy levees, which, though they may be built higher, are still generally conceived as earthen structures. Katrina has taught that the long-term reliability of such structures cannot be assured given the foundation conditions, if known, and the degree of surge and wave exposure that must now be assumed.

The experience with the Chalmette levee along the south bank of the MRGO is instructive. The reasons that it failed can be debated, whether poor design or construction, but the result is indisputable. Ultimately, the USACE found that it could not retain this earthen structure at the design grade, however inadequate that may have been, despite two major augmentations in the late 1970s and mid-1980s, and a less extensive rebuilding in the early 1990s. These construction efforts, like the recent repair projects, raised the levee crown to 20 feet, incorporating a 2.5 foot overbuild, to account for settlement. Efforts to improve reliability by armoring the recently rebuilt MRGO levee have been hampered by awareness that the new embankment will require additional lifts at relatively short intervals to counteract erosion and settlement, before it reaches a less dynamic condition. This remediation would be complicated by the need to repeatedly remove and replace armor installed on the levee slopes and crown.

The only structures that survived on the Chalmette levee run were the water control structures at Bayous Bienvenue and Dupre, which were left by the storm as islands of solidity in a sea of destruction. One of the most important lessons of Katrina was that pile-supported structures like these, as well as T-walls used sparingly elsewhere in the GNO HPS, were capable of surviving the worst that Katrina could deliver.

Team Louisiana members have not been directly engaged in the planning for Category 5 protection now in progress through the Louisiana Coastal Protection and Restoration study (LACPR). We understand that some USACE-IPET investigators have been engaged for this work, so it is hoped that information derived from study of Katrina levee failures has found its way into this planning initiative. The proposed alignment options that have been publicly distributed for the east side of New Orleans all appear to be land-based and, for the most part, follow existing levee rights-of-way (http://www.lacpr.usace.army.mil/publications/03-28-2006/South%20East%20LA%20Levees%20Alignments.pdf). The geometry of the funnel does not appear to have been addressed and no detail has yet been provided on the types of structures proposed.

Proposals for more reliably protecting the eastern side of New Orleans can be derived from study of the GNO HPS failures during Katrina. One would be to construct a pile-supported structure similar to that used by the Dutch for the Oostershelde closure across the 'funnel' (Figure 206 and 207). The Oostershelde closure is an elevated causeway supported by concrete piers providing guides for vertically sliding closure gates. Such a structure could be constructed modularly across shallow state-owned water bottoms generally less than 15 feet deep. There, it would be free of most right-of-way limitations and could be positioned to avoid deep tidal passes. While the length of the structure, depending upon the route taken, could be substantial, 40 miles or more, each module would be a far less massive than that of the Oostershelde which crosses waters averaging 50 feet in depth (Figure 8.4).

Such a structure might have a higher initial cost than the longer levee runs and integrated tidal pass gates suggested in the proposed Corps of Engineers LACPR alignments Federal Government Studies. On the other hand, reliability would be greatly enhanced if the surge barrier can be constructed to avoid most of the long-term maintenance problems that afflict gravity-supported flood protection structures periodically exposed to hurricane surge and waves. A road following the structure would be necessary for servicing but might also serve as an additional hurricane evacuation route. Furthermore, such a system might have a less deleterious effect on the environment, as it could be constructed to minimize wetland impacts and would interrupt hydrologic exchange only when storms threaten, rather than every day. Very importantly, given past history, such a structure could not be used to promote pumped drainage of the remaining wetlands.

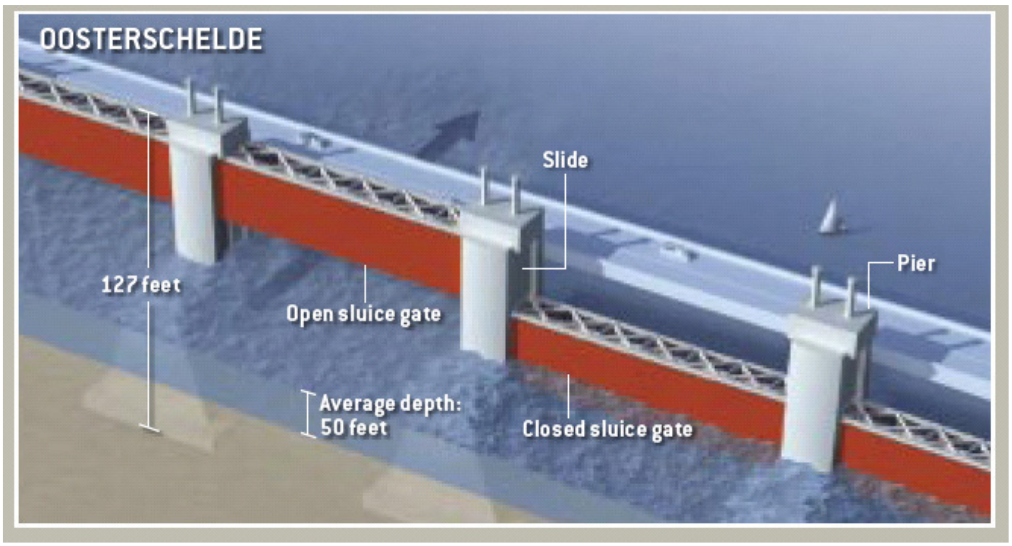


Figure 206a Schematic of a section of the Oosterschelde closure structure built in water depths 50 feet deep showing gates in open and closed positions.

Figure 206b. Photo of section of the Oosterschelde Barrier in the Netherlands.

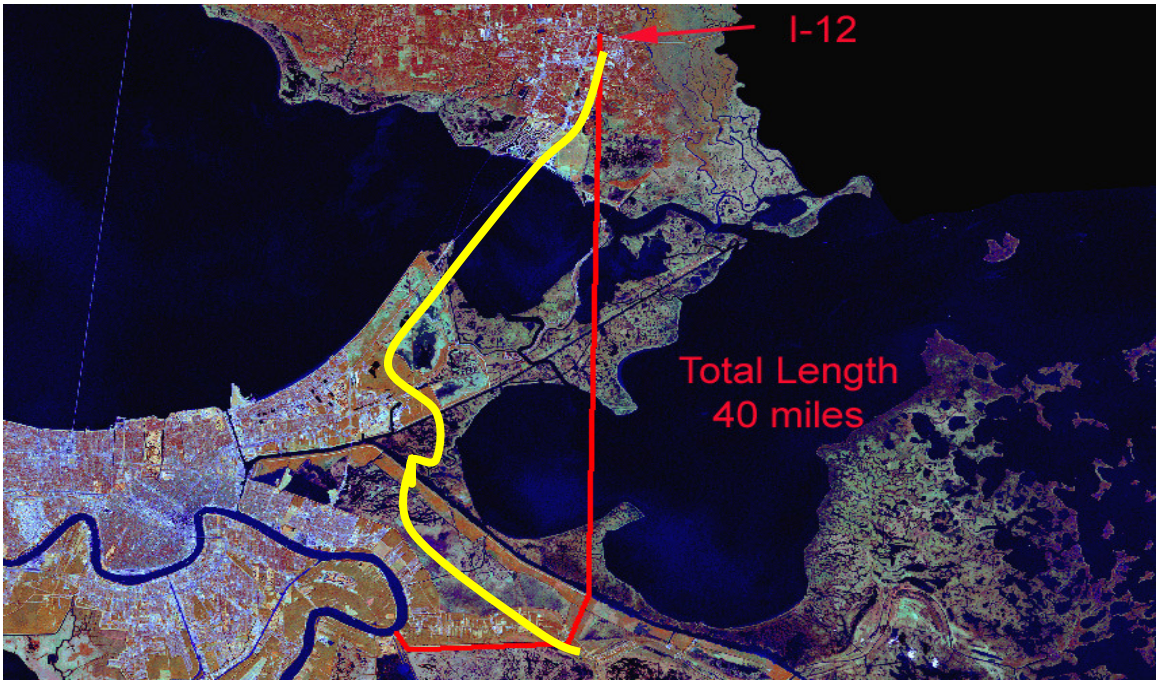


Figure 207. Possible alignments for a pile-supported structure to close the funnel east of New Orleans.

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REFERENCES

American Society of Civil Engineers (ASCE) and University of California at Berkeley. 2005 (November). *Preliminary Report on the performance of the New Orleans Levee Systems in Hurricane Katrina on August 29, 2005.* Report No. UCB/CITRIS – 05/01. Ver 1.2. Accessed September 20, 2006: http://www.asce.org/files/pdf/katrina/teamdatareport1121.pdf

Beach Erosion Board. 1954. TR-4 Shore Protection Planning and Design

Beach Erosion Board. 1973. Shore Protection Manual

Bligh, W.G. 1910. "Dams, Barrages and Weirs on Porous Foundations," Engineering News, p. 708.

Brouwer, G., 2003. The Creeping Storm. Civil Engineering-ASCE 73(6): 46-55.

Burk and Associates, Inc. 1990. London Avenue Interim Floodwalls and Levees. Revised General Design Memorandum, prepared for Board of Levee Commissioners of the Orleans Levee District, Contract No. 2049-0269, New Orleans, La.

Collins, S.M., 2006. Preparing for a Catastrophe: the Hurricane Pam Exercise. Opening statement of Senator Susan M. Collins, Chair, Senate Committee on Homeland Security and Governmental Affairs, January 24, 2006, Washington, D.C. (hsgac.senate.gov /_files/0124065MCOpen.pdf).

Chow, V.T. 1983. Open Channel Hydraulics, 20th Printing, 1983, McGraw-Hill Book Company, New York, NY. 680 pp.

Cunningham R., Gisclair D., Paper presented. 2003. ASPRS Topographic Mapping Symposium, Charleston, SC.

Cunningham R, Gisclair D. and J. Craig. 2004. The Louisiana Statewide LiDAR Project, (http://atlas.lsu.edu/central/la_lidar_project.pdf).

Eustis Geotechnical Reports

Eustis Engineering Co. 1986. Geotechnical Investigation for Orleans Levee District on London Avenue Outfall Canal (2 volumes). Report to Burk and Associates, Metairie, La.

Eustis Engineering Co. 1993. Additional Geotechnical Analyses for London Avenue Outfall Canal Proposed I-Walls and T-Walls, Mirabeau Avenue to Leon C. Simon Boulevard. Report to Burk-Kleinpeter, Inc., New Orleans, La.

Eustis Engineering Co. 1994. Geotechnical Investigation for London Avenue Outfall Canal Proposed Levee Crown Degrading, Mirabeau Avenue to Leon C. Simon Boulevard. Report to Burk-Kleinpeter, Inc., New Orleans, La.

Eustis Engineering Co. 1988. Geotechnical Investigation for Metairie Relief Canal (17th Street Canal), Orleans Levee Board Project 2043-0222. Report to Majerski and Masters, Inc., New Orleans, La.

Feyen, J.C., J.H. Atkinson and J.J. Westerink, 2002: GWCE-Based Shallow Water Equation Simulations of the Lake Pontchartrain - Lake Borgne Tidal System. Computational Methods in Water Resources XIV, S.M. Hassanizadeh et al. Eds., Volume 2, Developments in Water Science, 47, Elsevier, Amsterdam, 1581-1588.

GAO Reports

U.S. General Accounting Office. 1976. Cost, Schedule and Performance Problems of the Lake Pontchartrain and Vicinity Hurricane Protection Project, Report to Congress by Comptroller General, August 31, 1976, Washington, DC.

U.S. General Accounting Office. 1982. (don't seem to have in electronic form but available over web)

U.S. General Accountability Office. 2005. Army Corps of Engineers Lake Pontchartrain and Vicinity Hurricane Protection Project, Testimony before Subcommittee on Energy and Water Development, Committee on

Appropriations, House of Representatives, September 28, 2005, Washington, DC.

U.S. General Accountability Office. 2006. Strategic Planning Needed to Guide Future Enhancements Beyond Interim Levee Repairs, Report to Congressional Committees, September 2006, Washington, DC.

Graham, H.E. and Nunn, D. E. U.S. Weather Bureau and National Weather Service. 1959. *Meteorological considerations pertinent to standard project hurricane, Atlantic and Gulf Coasts of the United States*. Report No. 33, National Hurricane Research Project. U.S. Department of Commerce: Washington, DC. 76 pp.

Graumann, A., T. Houston, J. Lawrimore, D. Levinson, N. Lott, S. McCown, S. Stephens and D. Wuertz. 2005. Hurricane Katrina: a Climatological Perspective. NOAA National Climatic Data Center. Tech. Rept. 2005-01, October 2005, Asheville, NC. 27pp.

Harrison, Robert W., and Walter M. Kollmorgen. 1947. *Drainage reclamation in the coastal marsh-lands of the Mississippi River Delta.* The Louisiana Historical Quarterly 30:654-709.

Ho, F.P., J.C. Su, K.L. Hanevich, R.J. Smith and F.P. Richards. 1987. Hurricane Climatology for the Atlantic and Gulf Coasts of the United States. NOAA National Weather Service Technical Report NWS 38, Silver Spring, MD

Independent Levee Investigation Team (ILIT) 2006 (July). *Investigation of the performance of the New Orleans flood protection systems in Hurricane Katrina on August 29, 2005*. Final Report. Accessed September 20, 2006: http://www.ce.berkeley.edu/~new_orleans/

Innovative Emergency Management (IEM), Inc. 2004. *Southeast Louisiana catastrophic hurricane functional plan (draft).* IEM/TEC 04-070. A Report prepared for the Federal Emergency Management Agency. FEMA BPA HSFEHQ-04-A-0288, Task Order 001.

IEM Inc., 2004. Southeast Louisiana Catastrophic Hurricane Functional Plan, August 06, 2004, Innovative Emergency Management (IEM), Baton Rouge, Louisiana. 121pp.

Jackson, R.B. 1988. "E-99 Sheet Pile Wall Field Load Test Report." Technical Report No. 1. U.S. Army Engineer Division. Lower Mississippi Valley, Vicksburg, MS.

Jelesnianski, C.P. National Weather Service 1972. *SPLASH (Special Program to List Amplitudes of Surges from Hurricanes) I. Landfall Storms*, NOAA Technical Memorandum NWS TDL-46. National Weather Service Systems Development Office: Silver Spring, MD. 56 p.

Jelesnianski, Chester P., Jye Chen and Wilson A. Shaffer, National Weather Service. 1992. *SLOSH: Sea, Lake, and Overland Surges from Hurricanes*. NOAA Technical Report NWS 48. Silver Spring, MD. 71 p.

Kashef, A.I. 1986. *Groundwater engineering*. McGraw-Hill.

Knabb, R.D., Rhome J.R, and Brown, D.P. National Oceanic and Atmospheric Administration. National Hurricane Center. 2006 update. *Tropical cyclone report Hurricane Katrina 23-30 August 2005.* Accessed September 20, 2006: http://www.nhc.noaa.gov/pdf/TCR-AL122005_Katrina.pdf

Knabb, R.D., Rhome, J.R., and Brown, D.P., 2005. Tropical Cyclone Report: Hurricane Katrina. NOAA National Hurricane Center. 20 Dec., 2005. 42pp.

Kolb, C. R., Smith, F. L., and Silva, R. C. 1975. "Pleistocene sediments of the New Orleans-Lake Pontchartrain area," Technical Report S-75-6, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Kolb C.R., and Van Lopik, J.R. 1958. Geology of the Mississippi deltaic plain-southeastern Louisiana. USACOE Waterways Exp. Sta., Technical Report 2, 482 p.

Landsea, C. W., and W. M. Gray, 2002: "Florida's Coming Hurricane Calamities," *Miami Herald*, 22 July 2002

Lane, E.W. (1935)."Security from under-seepage-masonry dams on earth foundations," *Transactions American Society of Civil Engineering,* 100, pp. 1235-1351.

Luettich, R.A., Jr., and Westerink, J.J., 1995. Continental Shelf Scale Convergence Studies with a Barotropic Tidal Model, Quantitative Skill Assessment for Coastal Ocean Models, D. Lynch and A. Davies [eds.], Coastal and Estuarine Studies series, vol. 48, pp. 349-371, American Geophysical Union press, Washington, D.C.

Mashriqui, H.S., Kemp, G.P., van Heerden, I. Ll., Ropers-Huilman, B.D., Hyfield, E., Young, Y., Streva, K., and Binselam, 2006. Experimental Storm Surge Simulations For Hurricane Katrina, in "Coastal Hydrology and Water Quality", Xu, Y. J. & Singh, V. J., (editors), Proceedings of the AIH 25th Anniversary Meeting & International Conference, American Institute of Hydrology (AIH), May 21-24, 2006, Baton Rouge, Louisiana, USA. pp 481-489.

Mulcare, D. M. 2004. NGS Toolkit, Part 8: The National Geodetic Survey NADCON Tool, *Professional Surveyor magazine*, February 2004 Vol. 24, NO.2.

National Oceanic Atmospheric Administration. National Weather Service, Hydrometeorological Branch Office of Hydrology. 1972. *Preliminary revised standard project hurricane criteria for the Atlantic and Gulf Coasts of the United States*. Memorandum Hur 7-120. Silver Spring, MD, 43 p.

National Academy of Engineering & National Research Council. 2006. Third Report of NAE/NRC Committee on New Orleans Regional Hurricane Protection Projects, October 25, 2006. National Academies Press, Washington, DC.

Weather Service Reports

National Oceanic and Atmospheric Administration. 1972. Revised Standard Project Hurricane Criteria for the Atlantic and Gulf Coasts of the United States. Hydrometeorological Branch, Office of Hydrology, Memorandum HUR 7-120.

National Oceanic and Atmospheric Administration. 1979. Meteorological Criteria for Standard Project Hurricane and Probable Maximum Hurricane Windfields, Gulf and East Coasts of the United States. NOAA National Weather Service Technical Report NWS 23, Silver Spring, MD

Roberts, H.H., R.T. Beaubouef, N.D. Walker, G.W. Stone, S. Bentley, A. Sheremet, and I. van Heerden. 2002. Sand-rich bayhead deltas in Atchafalaya Bay (Louisiana): Winnowing by cold front forcing: Coastal Sediments '03, May 18-23, 2003, Clearwater Beach, Florida.

Saucier, R.T., 1963, Recent geomorphic history of the Pontchartrain Basin: Louisiana State University Press, 114 p.

Saucier, R. T., 1994, Geomorphology and Quaternary geologic history of the lower Mississippi valley: United States Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, v. 1, 364 p.

Seed R. Bea, R.G., Abdelmalak, R.I., Athanasopoulos, A.G., Boutwell, G.P., Bray, J.D., Briaud, I.L., Cheung, C., Cobos-Roa, D., Cohen-Waeber, J., Collins, B.D., Ehrensing, L., Farber, D., Hanemann, M., Harder, L.F., Inkabi, K.S., Kammerer, A.M., Karadeniz,D., Kayen, R.E., Moss, RES, Nicks, J., Nimmala, S., Pestana, J.M., Porter, J. Rhee, K., Riemer, MF, Roberts, K., Rogers, J.D., Storesund, R., Govindasamy, A.V., Vera-Grunauer, X., Wartman, J.E., Watkins, C.M., E. Wenk Jr., and S. C. Yim. 2005. "Preliminary Report on the Performance of the New Orleans Levee Systems in Hurricane Katrina on August 29, 2005" (Report No. UCB/CITRIS – 05/01, November 17, 2005).

Schwerdt, R.W., Ho, F.P., Watkins, R.R. National Oceanic and Atmospheric Administration, National Weather Service. 1979: *Meteorological criteria for standard project hurricane and probable maximum hurricane windfields, Gulf and East Coasts of the United States.* NOAA Technical Report NWS 23, U.S. Department of Commerce: Silver Spring, MD, 317 pp.

Shallat, T. 2000. In the wake of Hurricane Betsy. In *Transforming New Orleans and its Environs: Centuries of Change,* edited by C.E. Colten, 121-138. Pittsburgh: University of Pittsburgh Press.

Soil Conservation Service, USDA, 1968. SCS National Engineering Handbook.

Terzaghi, K., 1929, Effect of Minor Geologic Details on the Safety of Dams: in Geology and Engineering for Dams and Reservoirs, American Institute of Mining and Metallurgical Engineering, Technical Publication 215, pp. 31-44.

Thompson, E. F., and Cardone, V. J., 1996. Practical modeling of hurricane surface wind fields. Journal of Waterway Port Coastal and Ocean Engineering - ASCE 122(4): 195-205

Times-Picayune. 2002. "Washing Away" Special Report. Five Part Series, June 23-27. Accessed September 20, 2006: http://www.nola.com/hurricane/?/washingaway/

Turnbull, W. J., and Mansur, C. I. 1961. "Investigation of underseepage—Mississippi River levees." and "Design of underseepage control measures for dams and levees." Trans. Am. Soc. Civ. Eng., 126(1), 1429-1486.

USACE IPET Reports

US Army Corps of Engineers. Interagency Performance Evaluation Task Force (IPET).
2005 (December). *Summary of field observations relevant to flood protection in New Orleans, LA. Interim report to Task Force Guardian.* New Orleans Hurricane Protection Projects Data. Accessed September 20, 2006: https://ipet.wes.army.mil/NOHPP/_Post-Katrina/(IPET)%20Interagency%20Performance%20Evaluation%20TaskForce/Reports/ASCE_NSF%20Report%20Assessment_%20IPET_120505.pdf#search=%22.%20Summary%20of%20Field%20Observations%20Relevant%20to%20Flood%20Protection%20in%20New%20Orleans%22

---. 2006a (January). *Performance evaluation plan and interim status, report 1 of a series, performance evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System, draft final report*. New Orleans Hurricane Protection Projects Data. Accessed September 20, 2006: https://ipet.wes.army.mil/ (Folder Post-Katrina/(IPET) Interagency Performance Evaluation Taskforce/Reports/IPET Report 1.pdf)

---. 2006b (March). *Performance evaluation status and interim results, report 2 of a series, performance evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System* New Orleans Hurricane Protection Projects Data. Accessed September 20, 2006: https://ipet.wes.army.mil/ (Folder Post-Katrina/(IPET) Interagency Performance Evaluation Taskforce/Reports/IPET Report 2)

2006c (June). *Performance evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System. Draft Final Report of the Interagency Performance Evaluation Task Force.* New Orleans Hurricane Protection Projects Data, Accessed September 20, 2006: https://ipet.wes.army.mil/ (Folder Post-Katrina/(IPET) Interagency Performance Evaluation Taskforce/Reports/IPET Draft Final Report)

USACE As-Built Drawings

U.S. Army Corps of Engineers. 1983. IHNC East and West Levee and Citrus Back Levee Capping Floodwall, As-built Plans, Lake Pontchartrain and Vicinity Hurricane Protection, New Orleans District, New Orleans, La.

U.S. Army Corps of Engineers. 1994. London Avenue Outfall Canal, As-built Plans, Lake Pontchartrain High-Level Plan, New Orleans District, New Orleans, La.

U.S. Army Corps of Engineers. 1995. Orleans Avenue Outfall Canal, As-built Plans, Lake Pontchartrain High-Level Plan, New Orleans District, New Orleans, La.

U.S. Army Corps of Engineers. 1994. London Avenue Outfall Canal, As-built Plans, Lake Pontchartrain High-Level Plan, New Orleans District, New Orleans, La.

USACE 1984 RE-EVALUATION

U.S. Army Corps of Engineers. 1984. Lake Pontchartrain and Vicinity Hurricane Protection Project Reevaluation Study (3 volumes), New Orleans District, New Orleans, La.

USACE General Design Memoranda

U.S. Army Corps of Engineers. 1959. Mississippi River – Gulf Outlet, Channels, Miles 39.01 to 63.77, General Design Memorandum 1B, New Orleans District, New Orleans, La.

U.S. Army Corps of Engineers. 1957. Mississippi River – Gulf Outlet, Channels, Miles 63.77 to 68.85, General Design Memorandum 1A, New Orleans District, New Orleans, La.

U.S. Army Corps of Engineers. 1966. Chalmette Area Plan, General Design Memorandum 3, Lake Pontchartrain Barrier Plan, New Orleans District, New Orleans, La.

U.S. Army Corps of Engineers. 1967. Inner Harbor Navigation Canal West Levee, Florida Avenue to IHNC Lock, General Design Memorandum 2, Lake Pontchartrain Barrier Plan, New Orleans District, New Orleans, La.

U.S. Army Corps of Engineers. 1968. Chalmette Area Plan, Chalmette Extension, General Design Memorandum 3, Supplement 1, Lake Pontchartrain Barrier Plan, New Orleans District, New Orleans, La.

U.S. Army Corps of Engineers. 1968. Inner Harbor Navigation Canal Remaining Levees, General Design Memorandum 2, Supplement 8, Lake Pontchartrain Barrier Plan, New Orleans District, New Orleans, La.

U.S. Army Corps of Engineers. 1971. New Orleans East Back Levee, General Design Memorandum 2, Supplement 4, Lake Pontchartrain Barrier Plan, New Orleans District, New Orleans, La.

U.S. Army Corps of Engineers. 1987. Jefferson Parish/St. Charles Parish Return Levee, General Design Memorandum 17A, Lake Pontchartrain High-Level Plan, New Orleans District, New Orleans, La.

U.S. Army Corps of Engineers. 1988. Orleans Avenue Outfall Canal, General Design Memorandum 19 (3 volumes), Lake Pontchartrain High-Level Plan, New Orleans District, New Orleans, La.

U.S. Army Corps of Engineers. 1989. London Avenue Outfall Canal, General Design Memorandum 19A (2 volumes), Lake Pontchartrain High-Level Plan, New Orleans District, New Orleans, La.

U.S. Army Corps of Engineers. 1990. Seventeenth Street (Metairie Relief) Outfall Canal, General Design Memorandum 20 (2 volumes), Lake Pontchartrain High-Level Plan, New Orleans District, New Orleans, La.

USACE Design Manuals

U.S. Army Corps of Engineers. 1952, 1986, 1993. Engineering and Design for Seepage Analysis and Control for Dams. EM 1110-2-1901.

U.S. Army Corps of Engineers. 1970, 2003. Engineering and Design for Slope Stability. EM 1110-2-1902.

U.S. Army Corps of Engineers. 1978, 2000. Design and Construction of Levees. EM 1110-2-1913.

U.S. Army Corps of Engineers. 1994. Engineering and Design of Sheet Pile Walls. EM 1110-2-2504.

U.S. Bureau of Reclamation. 1974. "*Design of small dams: U.S. Department of the Interior, Bureau of Reclamation, Water Resources Technical Publication.*" 816 p.

van Heerden, I. Ll. 1994. *A long-term comprehensive management plan for Coastal Louisiana to ensure sustainable biological productivity, economic growth and the continued existence of its unique culture and heritage.* A Technical Report prepared for the Governor's Office of Coastal Activities. NSMEP-CCEER Report, Louisiana State University, 49pp. http://www.publichealth.hurricane.lsu.edu/ (Technical Reports)

van Heerden, I. 1999. A Comprehensive Management Plan for Coastal Louisiana. *ACTA GEOGRAPHICA SINICA* 54(2):165-17.

Van Heerden, I. 2003. "New Orleans – A City in Peril – Public Health Impacts of a Major Tropical Storm" Presented to the New Orleans Sewerage and Water Board and Corps of Engineers, Mayor's Environmental Breakfast. New Orleans January 16, 2003.

van Heerden, I. and S.A. Binselam, 2004a. Hurricane Pam Exercise (CD), prepared for the Louisiana OHSEP. Available online at http://hurricane.lsu.edu/floodprediction/PAM_Exercise04/

van Heerden, I., Diaz, J. and K. Streva. 2004b. Flooding New Orleans: A Public Health Catastrophe in the Making. Invited presentation to the CDC, August 2004.

van Heerden, I., Diaz, J. and K. Streva. 2005. "New Orleans and Hurricanes: A City in Peril." 14th World Congress on Disaster and Emergency Medicine, Edinburgh, Scotland, May 16-20. Talk and abstract, *Prehospital and Disaster Medicine 2005*; 20(2):s64-s65

van Heerden, I. with M. Bryan. 2006. "The Storm." Penguin Group, ISBN: 0670037818

van Heerden, I., Kemp, G.P., and Mashriqui, H.M. 2007a. "Use of the ADCIRC Storm Surge Model for Hurricane Katrina Surge Predictions and Levee Forensic Studies" GeoDenver 2007 conference proceedings, Feb 2007.

van Heerden, I and Kemp, G.P. 2007b. "A Conceptual Hurricane Surge Protection Plan for Coastal Louisiana" GeoDenver 2007 conference proceedings, Feb 2007.

van Zyl, D., and Harr, M. E. 1981. "Seepage erosion analysis of structures." *Proc., 10th Int. Conf. on Soil Mechanics and Foundation Engineering*, Vol. 1, A. A. Balkema, Rotterdam, The Netherlands, 503–509.

Weaver R.J., D.N. Slinn, *Coastal Eng. 2004, Proc. 29th Int. Conf.* 2**,** 1532 (2004).

Westerink, J.J., Feyen, J.C., Atkinson, J.H., Richard A. Luettich, R.A., Dawson, C.N., Powell, M.D., Dunion, J.P., Roberts, H.J., Kubatko, E.J., Pourtaheri, H., 2004. "A New Generation Hurricane Storm Surge Model for Southern Louisiana," University of Notre Dame, Notre Dame, IN 46556, http://www.nd.edu/~adcirc/index.htm.

Westerink, J.J., Luettich, R.A., Jr. and Muccino, J., 1994. "Modeling Tides in the Western North Atlantic Using Unstructured Graded Grids." *Tellus* 46a(2):178-199.

Yamazaki and Penland (contributors). 2002. "Environmental Atlas of the Lake Pontchartrain Basin." USGS, US EPA, UNO and Save Our Lake, Lake Pontchartrain Basin Foundation. Accessed October 3, 2006: http://pubs.usgs.gov/of/2002/of02-206/phy-environment/recent-hurricanes.html

APPENDIX 1. SAFFIR-SIMPSON HURRICANE SCALE

The Saffir-Simpson Hurricane Scale is a 1-5 rating based on the hurricane's present intensity. This is used to give an estimate of the potential property damage and flooding expected along the coast from a hurricane landfall. Wind speed is the determining factor in the scale, as storm surge values are highly dependent on the slope of the continental shelf and the shape of the coastline, in the landfall region. Note that all winds are using the U.S. 1-minute average.

Category One Hurricane:
Winds 74-95 mph (64-82 kt or 119-153 km/hr). Storm surge generally 4-5 ft above normal. No real damage to building structures. Damage primarily to unanchored mobile homes, shrubbery, and trees. Some damage to poorly constructed signs. Also, some coastal road flooding and minor pier damage. Hurricane Lili of 2002 made landfall on the Louisiana coast as a Category One hurricane. Hurricane Gaston of 2004 was a Category One hurricane that made landfall along the central South Carolina coast.

Category Two Hurricane: Winds 96-110 mph (83-95 kt or 154-177 km/hr). Storm surge generally 6-8 feet above normal. Some roofing material, door, and window damage of buildings. Considerable damage to shrubbery and trees with some trees blown down. Considerable damage to mobile homes, poorly constructed signs, and piers. Coastal and low-lying escape routes flood 2-4 hours before arrival of the hurricane center. Small craft in unprotected anchorages break moorings. Hurricane Frances of 2004 made landfall over the southern end of Hutchinson Island, Florida as a Category Two hurricane. Hurricane Isabel of 2003 made landfall near Drum Inlet on the Outer Banks of North Carolina as a Category 2 hurricane.

Category Three Hurricane:
Winds 111-130 mph (96-113 kt or 178-209 km/hr). Storm surge generally 9-12 ft above normal. Some structural damage to small residences and utility buildings with a minor amount of curtainwall failures. Damage to shrubbery and trees with foliage blown off trees and large trees blown down. Mobile homes and poorly constructed signs are destroyed. Low-lying escape routes are cut by rising water 3-5 hours before arrival of the center of the hurricane. Flooding near the coast destroys smaller structures with larger structures damaged by battering from floating debris. Terrain continuously lower than 5 ft above mean sea level may be flooded inland 8 miles (13 km) or more. Evacuation of low-lying residences with several blocks of the shoreline may be required. Hurricanes

Jeanne and Ivan of 2004 were Category Three hurricanes when they made landfall in Florida and in Alabama, respectively.

Category Four Hurricane:
Winds 131-155 mph (114-135 kt or 210-249 km/hr). Storm surge generally 13-18 ft above normal. More extensive curtainwall failures with some complete roof structure failures on small residences. Shrubs, trees, and all signs are blown down. Complete destruction of mobile homes. Extensive damage to doors and windows. Low-lying escape routes may be cut by rising water 3-5 hours before arrival of the center of the hurricane. Major damage to lower floors of structures near the shore. Terrain lower than 10 ft above sea level may be flooded requiring massive evacuation of residential areas as far inland as 6 miles (10 km). Hurricane Charley of 2004 was a Category Four hurricane made landfall in Charlotte County, Florida with winds of 150 mph. Hurricane Dennis of 2005 struck the island of Cuba as a Category Four hurricane.

Category Five Hurricane:
Winds greater than 155 mph (135 kt or 249 km/hr). Storm surge generally greater than 18 ft above normal. Complete roof failure on many residences and industrial buildings. Some complete building failures with small utility buildings blown over or away. All shrubs, trees, and signs blown down. Complete destruction of mobile homes. Severe and extensive window and door damage. Low-lying escape routes are cut by rising water 3-5 hours before arrival of the center of the hurricane. Major damage to lower floors of all structures located less than 15 ft above sea level and within 500 yards of the shoreline. Massive evacuation of residential areas on low ground within 5-10 miles (8-16 km) of the shoreline may be required. Only 3 Category Five Hurricanes have made landfall in the United States since records began: The Labor Day Hurricane of 1935, Hurricane Camille (1969), and Hurricane Andrew in August, 1992. The 1935 Labor Day Hurricane struck the Florida Keys with a minimum pressure of 892 mb--the lowest pressure ever observed in the United States. Hurricane Camille struck the Mississippi Gulf Coast causing a 25-foot storm surge, which inundated Pass Christian. Hurricane Andrew of 1992 made landfall over southern Miami-Dade County, Florida causing 26.5 billion dollars in losses--the costliest hurricane on record. In addition, Hurricane Gilbert of 1988 was a Category Five hurricane at peak intensity and is the strongest Atlantic tropical cyclone on record with a minimum pressure of 888 mb.

APPENDIX 2. THE STOPPED CLOCK PROGRAM

Background

Following an article in the New Orleans Times Picayune published 12 October 2005, New Orleans and St. Bernard residents whose homes had been flooded by the various levee breaches during Hurricane Katrina were encouraged to report stopped clock data and accounts of flooding to Team Louisiana field investigators. Residents were provided a contact number to the LSU Hurricane Center and asked to provide any relevant information to support investigators in their reconstruction of a timeline for flooding. The following report outlines the descriptions and data provided by various residents; provides a catalog of collected photos; and displays Geographic Information System (GIS) maps of reported stopped clock times.

Data Collection

Stopped clock times and eye witness reports of city flooding following Hurricane Katrina have been supplied to the LSU Hurricane Center and field investigators since mid-October 2005, with some residents supplying stopped clock photos taken during and/or after the event.

Stopped clock number 35, reading 11:07, found within a flooded Gentilly residence near Elysian Fields & Robert E. Lee (HPHC 2005).

The type of clocks sought in this aspect of the study included mainly battery-operated clocks, but grandfather clocks, certain electronic clocks and one atomic clock time were also noted. Additionally, numerous eye witness accounts were recorded from New Orleans and St. Bernard area residents who stayed in their homes throughout Hurricane Katrina, to help characterize the speed and manner of flooding.

Reported clock times and eye witness accounts were documented in a phone log or field list and later entered into a spreadsheet indicating address, time, type of clock, source of the data, and related information (Table A2.1).

Whenever possible, additional pertinent information (e.g., the height of the clock in the residence, which usually ranged from three to six feet above ground) and other supplemental data (e.g., notes on whether the residence was built on slab or raised, the type of structure, etc.) were included in the data collection.

Methods

Information on stopped clock times or flooding times (e.g., as reported by residents when they first noticed ponding) were imported into ESRI ArcGIS 9.1. These data were then geocoded and mapped using 2004 Tiger Roads data. Entries were assigned individual record (log-id) numbers, with additional detail and descriptions for each resident account available in a companion document (Table A2.2).

Reported clock times were coded as follows:

0=eye witness account
1=battery-operated
2=unknown clock type/unverifiable
3=grandfather clock
4=atomic clock/other

Clock times assigned code "2" were unable to be verified from follow-up contact with residents who reported the time or left a message with the Hurricane Center, but who did not supply call back information, supplied a non-working return number, or could not otherwise be contacted again following receipt of the information.

Clock times were viewed in a GIS with overlay map layers such as levee shapefiles (with noted locations of the 17th Street Canal and London Ave. canal breaches), roads, extent of flooding on 31 August 2005 (from available NOAA shapefiles) and 1 September 2005 georeferenced Ikonos imagery supplied by the LSU Coastal Studies Institute (Figures A2.1-7). Pre-Katrina LIDAR levee height data were supplied by Y.S. Yang, LSU Hurricane Center.

Results

The following figures and tables outline the stopped clock and eye-witness datasets.

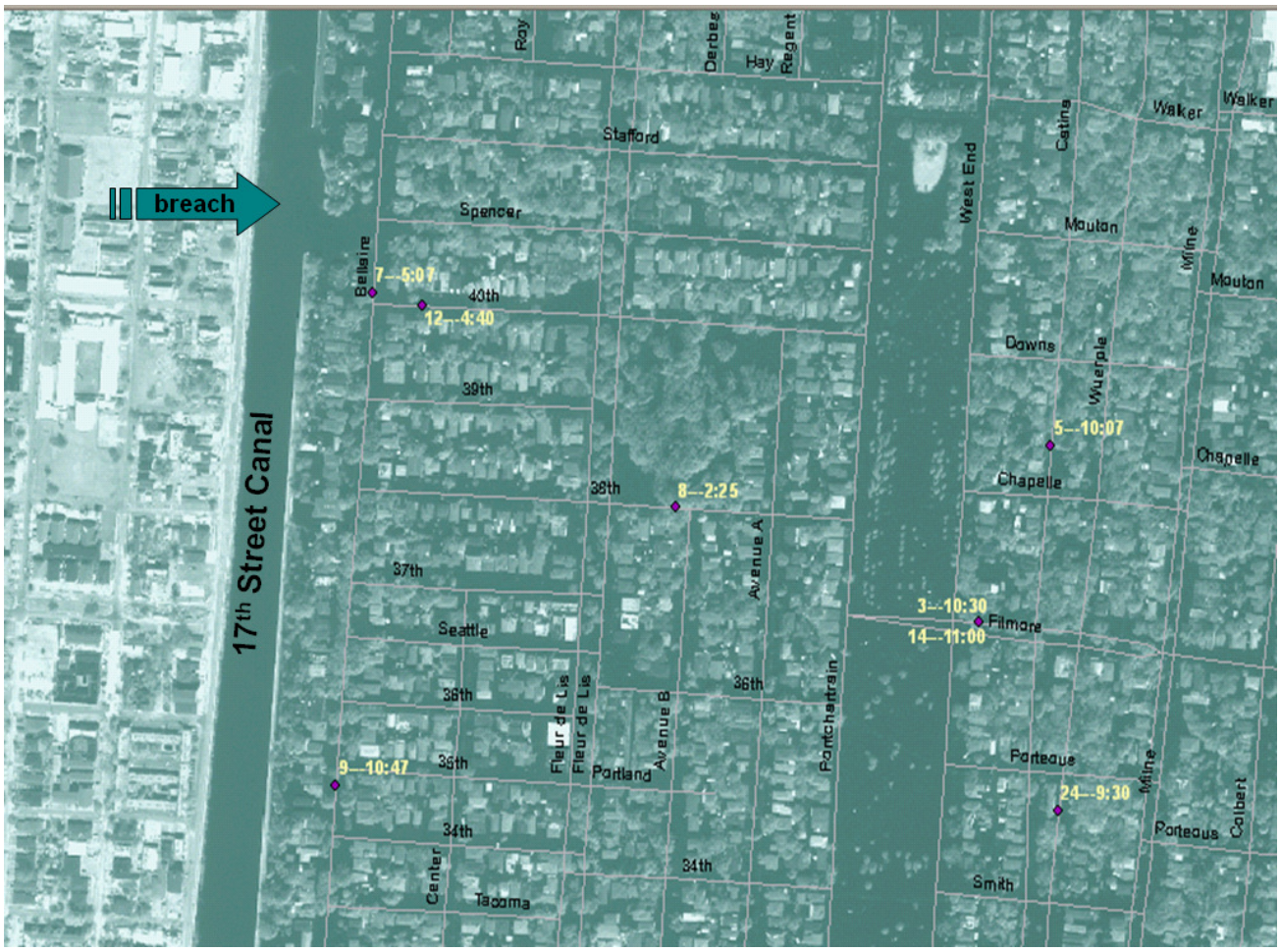

Figure A2.1. GIS snapshot of stopped clock times near the 17th Street canal levee breach (ESRI ArcGIS 9.1; 2005 Ikonos imagery-CSI; 2004 Tiger Roads data). CSI maximum flood depth LIDAR datasets indicate that this area was flooded between 6-12 feet of water.

Table A2.1. Reported clock times with general address and clock type.

LOG ID	General Street Address	Clock Time	Clock Type	Source of Data
1	Memphis St	12:45	3	KS_HCPHONELOG
2	Orleans Ave	10:00	0	KS_HCPHONELOG
3	Filmore	10:30	3	KS_HCPHONELOG
4	Pratt Dr	10:30	2	KS_HCPHONELOG
5	Catina	10:07	3	KS_HCPHONELOG
6	General Haag	5:34	1	KS_HCPHONELOG
7	Bellaire Dr	5:07	1	KS_HCPHONELOG
8	38th St	2:25	1	KS_HCPHONELOG
9	Bellaire Dr	10:47	1	KS_HCPHONELOG
10	Bellaire Dr	5:15	4	KS_HCPHONELOG
11	Orleans Ave	6:15	1	KS_HCPHONELOG
12	40th St	4:40	1	KS_HCPHONELOG
13	22nd St	11:15	1	KS_HCPHONELOG
14	Filmore	11:00	1	KS_HCPHONELOG
15	Charlotte Dr	9:00	0	KS_HCPHONELOG
16	18th St	4:15	1	KS_HCPHONELOG
17	22nd St	10:50	1	KS_HCPHONELOG
18	28th St	3:20	1	EZRAONSIT
19	Lane Ave	3:55	1	EZRAONSIT
20	Lane Ave	6:00	1	EZRAONSIT
21	Music St	6:17	1	EZRAONSIT
22	Egania	8:24	1	EZRAONSIT
23	Florida Ave	7:32	1	EZRAONSIT
24	Wuerple	9:30	0	KSFOLLOWUP
25	Reynes	5:00	2	WSFIELD
26	Reynes	7:45	2	WSFIELD
27	Forestall	5:30	2	WSFIELD
28	Read Blvd	10:30	0	VAN HEERD
29	Pratt	11:15	1	WSFIELD
30	Pratt	9:04	2	WSFIELD
31	Pratt	10:30	2	WSFIELD
32	Pratt	7:53	2	WSFIELD
33	Wildair	6:16	2	WSFIELD
34	Wickfield	7:45	2	WSFIELD
35	New York	11:07	2	EZRAONSIT
36	Coventry	10:05	2	EZRAONSIT
37	Woodbine Rd	6:37	3	EZRAONSIT
38	Center	11:30	1	EZRAONSIT
39	Margaret St	9:48	1	EZRAONSIT

Figure A2.2. Stopped clock times near the London Ave. canal levee breaches (ESRI ArcGIS 9.1; 2005 Ikonos imagery-CSI; 2004 Tiger Roads data). CSI maximum flood depth LIDAR datasets indicate that residences near the west breach saw at least 8 feet of water.

Figure A2.3. Zoomed in view of the 17th Street Canal Breach (ESRI ArcGIS 9.1; 2005 Ikonos imagery-CSI; 2004 Tiger Roads data).

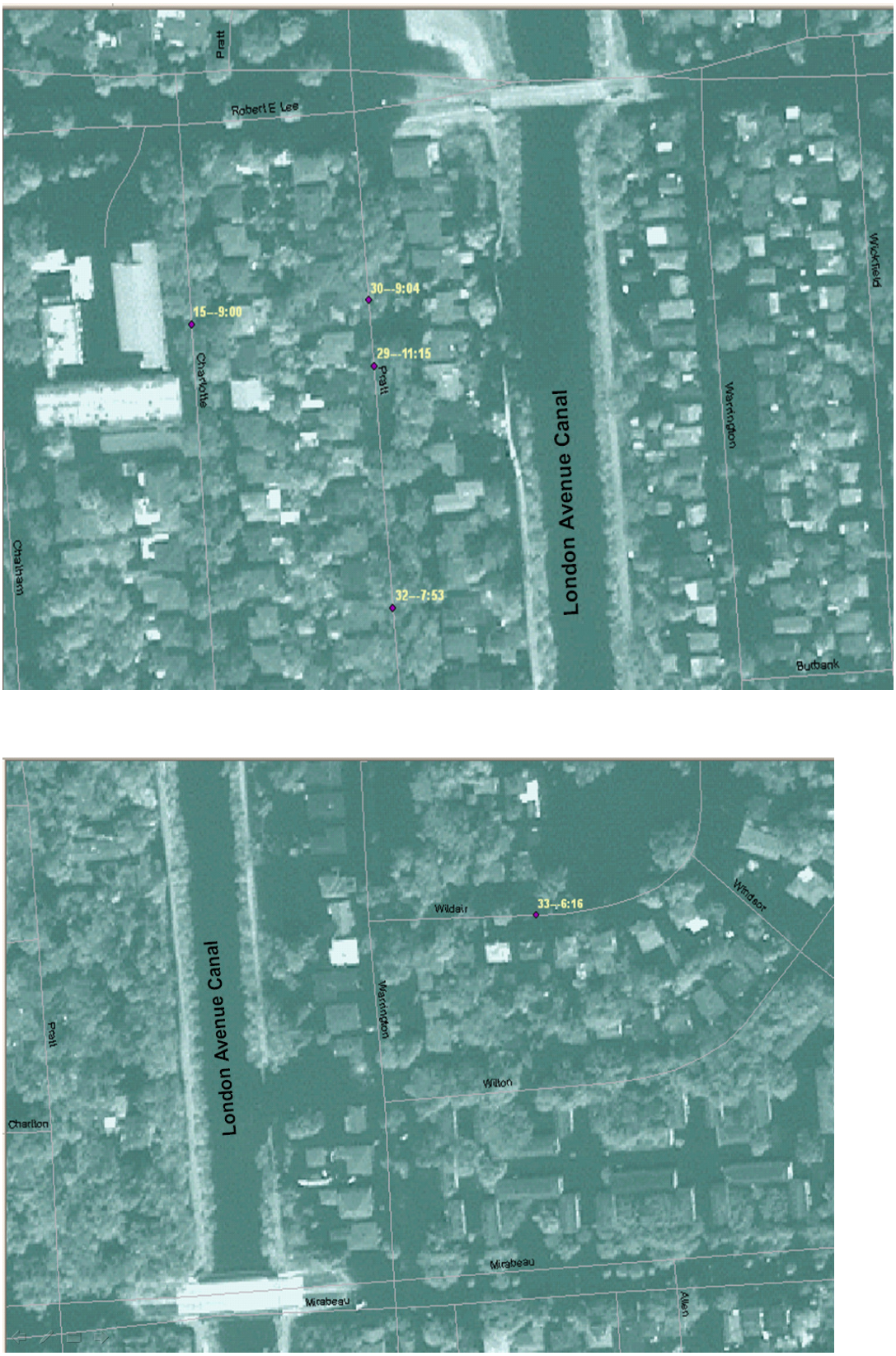

Figure A2.4. Zoomed in views of the London Street Canal Breaches, west and east, respectively (ESRI ArcGIS 9.1; 2005 Ikonos imagery-CSI; 2004 Tiger Roads data).

Figure A2.5. Stopped clock times near the Industrial canal levee breach (ESRI ArcGIS 9.1; 2005 Ikonos imagery-CSI; 2004 Tiger Roads data). CSI maximum flood depth LIDAR datasets indicate that residences here experienced well over 8 feet of water, with additional evidence suggesting high velocity flows (Mashriqui, AIH 2006, presentation). Note the severe displacement of residences.

Figure A2.6. Zoomed in view of the Industrial Canal breach (ESRI ArcGIS 9.1; 2005 Ikonos imagery-CSI; 2004 Tiger Roads data).

Table A2.2: Companion table of notes on each reported clock time

LOG ID	General Street Address	Clock Time	Clock Type	NOTES
1	Memphis St	12:45	3	water not in street at 9:30 but started to rise about 10, started coming into house around 11 (were still in house), filled up quickly, he was calling 911 around noon, whole house was completely under water by 12:30 – 1 PM.
2	Orleans Ave	10:00	0	
3	Filmore	10:30	3	the electronic clocks stopped at 5:30 (assuming Monday morning); grandfather clock, assumes water stopped pendulum at 10:30 next morning or night; about a half mile from the levee break
4	Pratt Dr	10:30	2	
5	Catina	10:07	3	
6	General Haag	5:34	1	clock was on waist high table
7	Bellaire Dr	5:07	1	right next to levee break in 17th St Canal. 5-6' on wall.
8	38th St	2:25	1	
9	Bellaire Dr	10:47	1	7.5 feet up in the kitchen close to water line
10	Bellaire Dr	5:15	4	atomic clock
11	Orleans Ave	6:15	1	clock was in kitchen, they had about 10 feet of water, plus they are up three steps.
12	40th St	4:40	1	2-3 battery operated watches all said 4:40. Says they are right next to the 17th street canal levee break.
13	22nd St	11:15	1	
14	Filmore	11:00	1	battery operated clock (8 feet up a wall) shut down at 11, morning of the 30th?
15	Charlotte Dr	9:00	0	lives directly in front of breach. Was there when London Ave levee failed. Says levee not overtopped; no water on Lakeshore Dr. south of lake. Evacuated to UNO (mostly dry). Said water rose, went down, then rose again. Monday between 9AM-12PM.
16	18th St	4:15	1	5' on wall, house 1' off ground (so 6' flooding at least).
17	22nd St	10:50	1	
18	28th St	3:20	1	
19	Lane Ave	3:55	1	
20	Lane Ave	6:00	1	
21	Music St	6:17	1	
22	Egania	8:24	1	
23	Florida Ave	7:32	1	
24	Wuerple	9:30	0	
25	Reynes	5:00	2	
26	Reynes	7:45	2	
27	Forestall	5:30	2	
28	Read Blvd	10:30	0	Methodist Hospital Generator
29	Pratt	11:15	1	
30	Pratt	9:04	2	
31	Pratt	10:30	2	
32	Pratt	7:53	2	
33	Wildair	6:16	2	
34	Wickfield	7:45	2	
35	New York	11:07	2	
36	Coventry	10:05	2	
37	Woodbine Rd	6:37	3	
38	Center	11:30	1	
39	Margaret St	9:48	1	

Conclusions

St. Bernard
Ivor: Several reports from southeast part of parish indicate flooding really started at 0830 when 40 Arpent Canal overtopped. 0615 Water comes in from Caernarvon levee to Poydras.

Lower 9th Ward
Ivor: Several clocks close to the breach stopped at 0745. First police calls were received at 0755 and were relayed to Bob Turner who called the NWS at 0810 who reported it at 0817. Residents in a 2-story reported flooding from the east at 0630 and then all hell broke loose at 0750. Various clocks stopped between 0743-0747, corresponds to observations of National Guard officers at Jackson Barracks.
WS: Otley at 0615. 0500 on clock 7 ft off ground at Reynes (clock 25) 3 blocks from IHNC just north of Claiborne. 0745 on clock 6 ft off ground in highest house in neighborhood at Reynes (clock 26) north of Claiborne. Report of water at 0530 to 0600 at Forestall (clock 27), one block east of Reynes.
Streva: 0824 on grandfather clock at Egania (clock 22) 7 blocks east of Lock between St. Claude and Claiborne. 0732 at Florida Ave. (clock 23), 7 blocks east of IHNC on 40 Arpent Levee

Orleans East
Reached high enough to stop generator at Methodist hospital at 10:30 (clock 28).

London Ave
Ivor: Several clocks at Mirabeau houses close to breach were flooded at 9:27 to 9:37. Video collected at 10:30 shows water up about 5 ft 2 blocks from breach. At breach, water is up to eaves of houses.
WS: Filmore break, 1115 to 1117 on several battery operated clocks at Pratt (clock 29). 0904 at Pratt (clock 30) v. close to Filmore breach, 1030 at Pratt Dr.(clock 31), 0753 (probably PM) on kitchen clock at Pratt (clock 32), 0808 (probably PM) next door in back. Woman at Paris and Bertha on lakefront fill between London Ave. and B. St. John saw water on sidewalk at 1700, in house at 2000. Mirabeau break: 0616 (probably PM) at Wildair (clock 33) 1 block from Mirabeau break near park. 0745 (probably PM) on Wickfield (clock 34) on east side between Mirabeau and Filmore.

Streva: 0900 Eyewitness reports break on west side from Charlotte (log id 15), 1 block away. 1030 on west side just north of Filmore about 6 blocks south of breach. 0930 Eyewitness reports water 2 blocks west of north break at Wuerple (log id 24). 0617 (probably PM) on Music St. (clock 21) 19 blocks east of Mirabeau break

17th St.

Ivor: Time-stamp on cell phone records for 4 calls for water in street 4 blocks from breach before 10:00. No water at 0937. Firemen say that breach occurred at 0927. Several low clocks in Robert E. Lee area stopped at 1040.

WS: Fellow in Marina Towers on east side of canal estimates that water started coming up between 0945 and 1100. Said it came up 7 ft in 40 minutes.

Streva: 1245 (probably PM) on grandfather clock at Memphis (clock 1) in Lakeview on west side of Orleans Canal. 1115 at 22nd St. (clock 13) one block east of 17th St. Canal 5 blocks north of Veterans. 1100 at Filmore Ave.(clock 14), 6 blocks east of Canal, 1030 on grandfather clock at same location (clock 3). 1050 at 22nd St. (clock 17) one block east of 17th St. Canal 5 blocks north of Veterans. 1047 at Bellaire (clock 9) along canal 7 blocks south of breach. 1007 at Catina (clock 5) 6 blocks east of breach. 1000 Eyewitness sees water at Orleans Ave. (log id 2), 19 blocks east of canal on west side of Orleans Canal. 0930 Eyewitness sees water at Wuerple (log id 24) 8 blocks east of breach. 0616 (probably PM) at Orleans Ave (clock 11), 19 blocks east of canal on west side of Orleans Canal. 0600 (probably PM) at Lane (clock 20), 19 blocks east of canal on west side of Orleans Canal. 0534 (probably PM) at General Haag (clock 6) 19 blocks east of canal on west side of Orleans Canal near Veterans. 0515 (probably PM) on canal 14 blocks south of breach. 0507 (probably PM) at Bellaire Dr. (clock 7), 4 blocks south of break. 0440 (probably PM) at 40th St. (clock 12) 3 blocks south of break. 0415 (probably PM) at 18th St. (clock 16) one block from canal but 21 blocks south of break. 0355 at Lane St. (clock 19), 8 blocks from canal. 0320 at 28th St. (clock 18), 15 blocks south of break. 0225 38th St. (clock 8), 5 blocks from break.

Data Review

For this study, possible sources of error may be introduced with all types of collected clock data as well as eye witness accounts. First, stopped clock times will not indicate AM or PM. Second, while some reported clock times were verifiable with photos, most clock times and indication of first flooding were reported in good faith, and could in some cases be suspect. Third, slight time variations exist and must be noted, in that eye witness accounts generally reported times when ponding was first noticed, while reported battery-operated clocks likely reflected the time water reached the clock and shorted out the batteries, while grandfather clock times were likely reflective of when floodwaters reached the pendulum, all clocks at varying heights. All times used in the analysis also build on the assumptions that clocks were accurately set and keeping correct time at the time of the storm (particularly in cases of neglected clocks or clocks which are difficult to reach or set, e.g., which may not have been adjusted to daylight savings, or ran fast or slow, etc. Finally, of particular note, other clock times, such as from the atomic clock and other battery-operated clocks running on electricity as well,

may have introduced other highly variable times when electricity was lost across city grids.

Figure A2.7a. All clock locations by LOG ID, New Orleans east and Industrial Canal (ESRI ArcGIS 9.1; 2005 Ikonos imagery-CSI; 2004 Tiger Roads data).

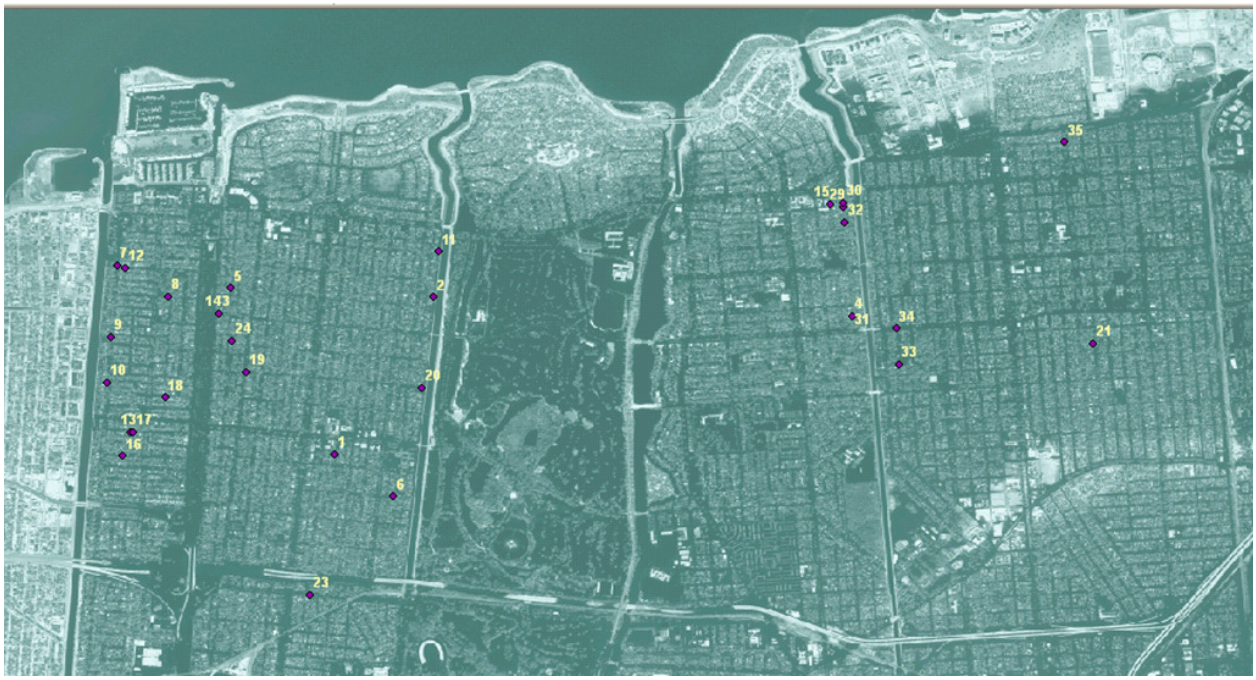

Figure A2.7b. All clock locations by LOG ID, New Orleans near 17th Street and London Avenue Canal breaches (ESRI ArcGIS 9.1; 2005 Ikonos imagery-CSI; 2004 Tiger Roads data).

Collected Photos

Grandfather Clock (clock 5) reading 10:07, from a flooded residence on Catina Street

Grandfather Clock (clock 1) reading 12:45, from a flooded residence on Memphis Street

Stopped clock (clock 18) reading 3:20, 28th Street Street

Stopped clock (clock 19) reading 3:55, Lane

Lane Ave, stopped clock at 6:00 (clock 20).

Egania (clock 22) reading 8:24.

Stopped clock on Florida Street (clock 23) reading 7:32.

Margaret Street (clock 39) 9:48.

Stopped clock on Music Street (clock 21) reading 6:17.

Center Street (clock 38) 11:30

Woodbine, stopped clock at 6:37 (clock 37)

APPENDIX 3. CURRENT REPORT OF DECEASED AND MISSING

The State Medical Examiner, Dr. Louis Cataldie, has spearheaded the Department of Health and Hospitals (DHH) effort to reunite the living and recover and identify the deceased. Initially, the Disaster Mortuary Operational Response Team (DMORT) and local parish coroners worked in conjunction with the State Medical Examiner's office. However, on March 1, DMORT ceased operation. The $17 million dollar morgue and identification center was closed less than four months after it opened. Following the closure of the DMORT facility, responsibility for victim identification has been returned to the Orleans parish coroner (who typically provides services for other parishes in the metropolitan region). First opened on September 7, 2005, the Family Assistance Center (FAC) handled reports of missing persons. Out of an initial 13,400 people reported missing, the FAC resolved all but 140 cases when it closed on August 1, 2006. Most recently, demolition workers uncovered the remains of presumed flood victim on December 2, 2006 in the damaged home in the Lower Ninth Ward.

DHH provides the official figures on dead and missing on their "Katrina Missing" website (http://www.dhh.louisiana.gov/offices/?ID=192). Last updated August 2, 2006, for Louisiana residents, their website reported 1,464 confirmed deceased victims and 135 persons reported missing. Of the confirmed dead, 1,118 victims perished within Louisiana, while 346 evacuees died outside of the state of Louisiana. Of the 135 persons officially recorded as missing, many are believed to be dead. For many of these victims, a body will never be recovered. It is likely, however, that these figures under-represent the true loss-of-life in Louisiana due to Hurricane Katrina and the subsequent levee failures. While all States received evacuees, only 32 have provided information on evacuee deaths within their states. Additionally, the State Medical Examiner, due to operational limitations, set October 31 as a deadline for a death to be considered Katrina related. Certainly, some number of Louisiana residents died because of circumstances related to Hurricane Katrina after October 31, 2005. Finally, in mid-November 2006, Orleans parish coroner Frank Minyard told the City Council that the Michoud area in New Orleans had never been searched for bodies.

Given these uncertain circumstances, all figures and maps contained within this appendix are preliminary and subject to revision.

A.3.1 Recovery Locations Dataset

In mid-October, the LSU Hurricane Center established a collaborative effort with the State Medical Examiner's office. As part of this collaboration, DHH provided data on the recovery locations for the deceased victims. The most recent dataset, obtained on September 14, 2006, lists the recovery locations of 863 victims throughout the State of Louisiana. Not listed are the 243 victims recovered by parish coroners. The recovery locations dataset corresponds to 77 percent of the total number of victims recovered within the State. Of the 863 victims listed in the dataset, recovery locations for 771 have been entered into a spatial database (geocoded). This corresponds to 69 percent of the victims recovered within the state.

The dataset includes the following information: Date of Recovery, Type of Facility, Facility Name, Street Address, Nearest Cross Street, City, State, Zip, Parish, Comments, Transported By, and Scene. Each entry in the dataset describes the recovery of one victim. In some cases, multiple victims were recovered from a single location. This current dataset is based on "Receipts of Remains" provided to DHH by the victim recovery teams. It is worth noting that the recovery of victims was performed by teams from numerous agencies. Coordination was poor and record keeping incomplete. As a result, the available data on the Katrina related fatalities remains imperfect and inconsistent.

The dataset indicates that most victims were recovered from Orleans parish, and most victims were recovered from residences. Of the 772 recoveries for which a parish is listed, nearly 72% list Orleans parish, followed by nearly 14% in St. Bernard parish. 6% victims were recovered from East Baton Rouge parish and 3% from Jefferson parish. The dataset lists a structure type for 774 victims. Of these, 52% were recovered from residences, 18% from hospitals and other health care facilities, 9% from nursing homes, 8% from street locations or other public places, 2% from the Convention Center, 2% from the Airport, and 1% from the Superdome. Only three drowning victims were recovered from outside the GNO HPS. The victims recovered from within GNO were distributed unevenly across the inundated area (Figure A3.2)

As noted, 771 of the recovery locations have been geocoded. Some of the tables below were derived by merging the recovery locations shapefiles with other shapefiles. Parish shapefiles were obtained from the LA GIS Data CD.

Table A3.1 Victim Recovery by Parish (based on information provided in dataset)

Parish	Recoveries	Percent of Known
ALEXANDRIA	1	0.13
ASCENSION	3	0.39
EAST BATON ROUGE	47	6.09
EAST FELICIANA	1	0.13
IBERVILLE	1	0.13
JEFFERSON	26	3.37
LAFAYETTE	13	1.68
MOREHOUSE	2	0.26
ORLEANS	555	71.89
OUACHITA	1	0.13
POINTE COUPEE	1	0.13
RAPIDES	2	0.26
ST JAMES	1	0.13
ST. BERNARD	106	13.73
TANGIPAHOA	4	0.52
TERREBONNE	8	1.04
UNKNOWN	91	
Grand Total	863	
Total Known	772	

The New Orleans Community Data Center provided shapefiles for planning districts and neighborhoods in Orleans parish. For St. Bernard parish, the US Census provides a polygon shapefile depicting the delineation of places (or communities) with the parish. Using these shapefiles, we can look at the distribution of fatalities within the two hardest hit parishes (Figure A 3.3). However, note that since the Census Places shapefile does not cover all of St. Bernard parish, the derived maps to not represent all victims recovered within this parish.

A3.3 Causes and Circumstances

It can be seen that the victims that have been recovered were not uniformly distributed throughout the area inundated. The Lower 9th Ward contributed many more victims than any other neighborhood. Beyond this observation, very little can presently be inferred about the causes and circumstances of those who died during Hurricane Katrina. Operationally, DHH uses a broad definition of deceased individuals for whom they provided identification and other services: anyone from the affected parishes that died between August 28 and October 31

for which the circumstances of death can be linked to Hurricane Katrina. For some cases, this link can be very tenous; for example, a baby, displaced by the flooding, that drowned in a hotel in Houston. Autopsies have been completed only in a handful of cases and these results have not yet been made public.

Table A3.2 Victim Recoveries by Neighborhood in Orleans Parish and Place In St. Bernard Parish

Neighborhood/ Place	Total Victims Recovered	Neighborhood/ Place	Total Victims Recovered
Lower Ninth Ward	73	Plum Orchard	6
Chalmette	44	Milan	5
Freret	35	Read Blvd West	5
Mid-City	24	Treme'/Lafitte	5
Milneburg	23	Bywater	4
Little Woods	21	Pines Village	4
West Lake Forest	21	B.W. Cooper	3
Lower Garden District	20	Desire Area	3
Central City	18	French Quarter	3
Lakeview	18	Dixon	2
Touro	17	East Carrollton	2
Seventh Ward	16	East Riverside	2
Arabi	16	Florida Dev	2
Fillmore	14	Gert Town	2
St. Anthony	14	Iberville Dev	2
Central Business District	13	Marlyville/Fountainebleau	2
Meraux	13	Village de l'est	2
Hollygrove	12	West Riverside	2
Viavant/Venetian Isles	12	Audubon	1
Gentilly Terrace	11	Bayou St. John	1
Read Blvd East	11	Black Pearl	1
St. Claude	11	City Park	1
Tulane/Gravier	11	Fischer Project	1
Pontchartrain Park	10	Lake Catherine	1
St. Roch	10	Lakeshore/Lake Vista	1
West End	10	Lakewood	1
Violet	10	Marigny	1
Dillard	9	Navarre	1
Broadmoor	8	Old Aurora	1
Fairgrounds	8	St. Bernard Area	1
Florida Area	8	Whitney	1
Gentilly Woods	6	Poydras	1
Holy Cross	6	**Total**	**623**

In very rough terms, it appears that approximately 50% of the in-state fatalities could have been caused by drowning. Naturally, the 85 victims recovered from outside of Orleans, St. Bernard, Jefferson, St. Tammany, or Plaquemines parish are not likely to have drowned, since only these parishes flooded. Likewise, of the victims recovered within the flooded parishes, only 584 victims were recovered from within the flood zone. Of these, 23 victims recovered from residences were found in attics or on the second or third floor well above any flooding. This leaves approximately 561 fatalities that occurred as a direct result of exposure to flood waters. While we can assume that drowning was the dominant cause of death among the victims, some resulted from other causes of death such trauma due to impact with debris.

A3.4 The missing

When the FAC closed on August 1, 2006, the list of missing contained 135 people and these cases were turned over to the local law enforcement agencies. This number is down from the 13,400 persons reported to the call center when it began operations. From a previous list of missing, DHH had identified 400 as potential fatalities based on information such as an address in heavily flooded areas and a lack of contact with family members since the storm. Using this list of 400, the NOFD resumed victim recovery operations on March 4. They have since recovered 17 victims; the most recent victim was found on May 28. At the time many missing persons on the list of 400 were thought to be deceased. However, it is worth noting that many individuals on the list of 400 were found alive before the FAC closed and that local enforcement officials have found alive at least five people reported as missing since the FAC closed.

A3.5 Recovery locations and levee breaks

Proximity to a breech location partially determines the flow velocity of the flood waters. These flood hazard characteristics influence the survivability of a person trapped in the flood waters. As a first examination of the relationship between fatalities and flow velocity, we use buffer zones around the breach locations and count the number of recoveries per buffer zone. Three buffer zones were calculated for each of the breeches: 500 ft, ½ mile and 1 mile buffers. In this step, breeches which are located close to each other, such the two breeches in the Industrial Canal along the Lower Ninth Ward, were treated as one and given just a single buffer zone for the three widths. When a buffer zone crossed a canal, only fatalities on the breech side of the canal were counted. So, the buffer around the London Avenue North breech only includes victims recovered from the west side of the canal.

The tables below present the results of the buffer zone calculations. Only three victims were recovered from within 500 ft of a levee breech, two near the Industrial Canal and 1 near the London Avenue North Breech. Forty-nine victims were recovered within a half-mile of a levee breech. Of these, over two-thirds were found in areas flooded by the Industrial Canal breeches. 145 victims were recovered with one mile of a levee breech. Of these, just under half were found in areas flooded by the Industrial Canal breeches. These results further illustrate the deadly hydraulic consequences of the current levee configuration around the MRGO-Industrial Canal complex.

Table A3.3. Density of recovered persons within a radii of different sizes from breach locations

500 ft Buffer	
Label	**Fatalities**
17thStreet	
40ArpentCentral	
40ArpentEast	
40ArpentWest	
CSXEast	
CSXWest	
Industrial	2
Lakefront	
LondonNorth	1
LondonSouth	
Michoud	
MRGO	
STBSouth	

Half Mile Buffer	
Label	**Fatalities**
17thStreet	2
40ArpentCentral	
40ArpentEast	
40ArpentWest	
CSXEast	1
CSXWest	1
Industrial	32
Lakefront	1
LondonNorth	2
LondonSouth	10
Michoud	
MRGO	
STBSouth	

One Mile Buffer	
Label	**Fatalities**
17thStreet	11
40ArpentCentral	10
40ArpentEast	
40ArpentWest	12
CSXEast	2
CSXWest	4
Industrial	71
Lakefront	4
LondonNorth	7
LondonSouth	24
Michoud	
MRGO	
STBSouth	

A3.6 Victims in the Lower 9th Ward and The MRGO Surge Funnel

As previously noted, the Lower 9th Ward neighborhood of New Orleans suffered the most fatalities. Various factors potentially explain why, including substandard housing stock, possible lower evacuation rates, and possible delays in relief assistance. Elderly status alone likely does not explain this observation, as the Greater New Orleans Community Data Center notes the age structure of the Lower 9th Ward follows the rest of Orleans and that the percentage of elderly living alone is lower in this neighborhood when compared with the rest of Orleans parish.

In explaining the high number of fatalities in the Lower 9th Ward one has to consider the proximity of this neighborhood to the end of the MRGO navigational canal at the mid-section of the Industrial Canal. As discussed in previous sections, the present configuration of levees around Lake Borgne and

MRGO confines the surge to a narrow channel, and the ADCIRC storm surge simulation reveals the effects of this configuration: high velocity and highly destructive water flows down the MRGO into the heart of a densely packed urban area. Following the failure of the levees along the Industrial Canal, this hydraulic system pushed high flow velocity waters into the Lower 9th Ward destroying homes from many blocks and leaving little chances of survival for trapped residents. MRGO and its hydraulic effect of funneling the system surge into the Lower 9th Ward provides the dominant explanation for the elevated mortality observed there.

A 3.7 Real-Time, Remote Estimation of Flood Fatalities

During the emergency phase, the LSU Hurricane Center provided support for State and local efforts to protect the lives of those trapped in the flooded regions and at the various evacuation centers. As part of this effort, we were tasked to provide a real-time estimate of the number of expected flood fatalities. Early efforts suffered from a lack of reliable flood data and an accurate estimate of the population that did not evacuate. For example, we had heard reports that the entire levee along Lake Borgne had collapsed. This occurrence would have implied water depths of over 17 ft in region (based on tidal gauges outside the levee system that read 17 ft). Based on this belief, we estimated 7,000 fatalities in New Orleans East and a total fatality estimate of 10,000-20,000. However, by September 29, a more reliable estimate of 1,500-3,000 was developed that has held up well (Table 4.4).

Estimating the number of fatalities during a flood event requires three key pieces of information. First, the number of people exposed to the flood must be closely approximated. In practical terms, this is the number of residents living within the flooded region minus those who evacuate and those who adequately shelter-in-place. Second, we need to know the distribution of the flood depths. Both of these variables vary geographically, so a data-rich GIS is an important tool in flood fatality estimation. Finally, we must use a flood fatality function that is essentially a dose-response relationship to estimate the flood fatality rate based on the depth of water. Multiplying the flood fatality rate by the exposed population gives an estimate of the number of flood fatalities.

For better or worse, Hurricane Katrina provided an initial test of this approach to flood fatality modeling. Once accurate flood data was included in the analysis, the model produced results that have proven to be reasonably accurate. Naturally, given the complexity of the processes involved, particularly the effectiveness of search and rescue efforts, order-of-magnitude accuracy is about

all that can be expected. As this research moves forward, the accuracy of the model can be expected to improve and better flood fatality model will prove useful as America prepares for the impacts of global sea level rise and increased hurricane activity. The next time a major American city floods, it is hoped that flood fatality models will do more than estimate the loss of life, but, instead, will help response officials to save lives by identifying areas of greatest concern. Likewise, as Louisiana moves into the future and considers the best options for mitigating the flood hazard, the flood fatality model will help determine the levee configurations, land use decisions, and preparedness policies that minimize loss of life.

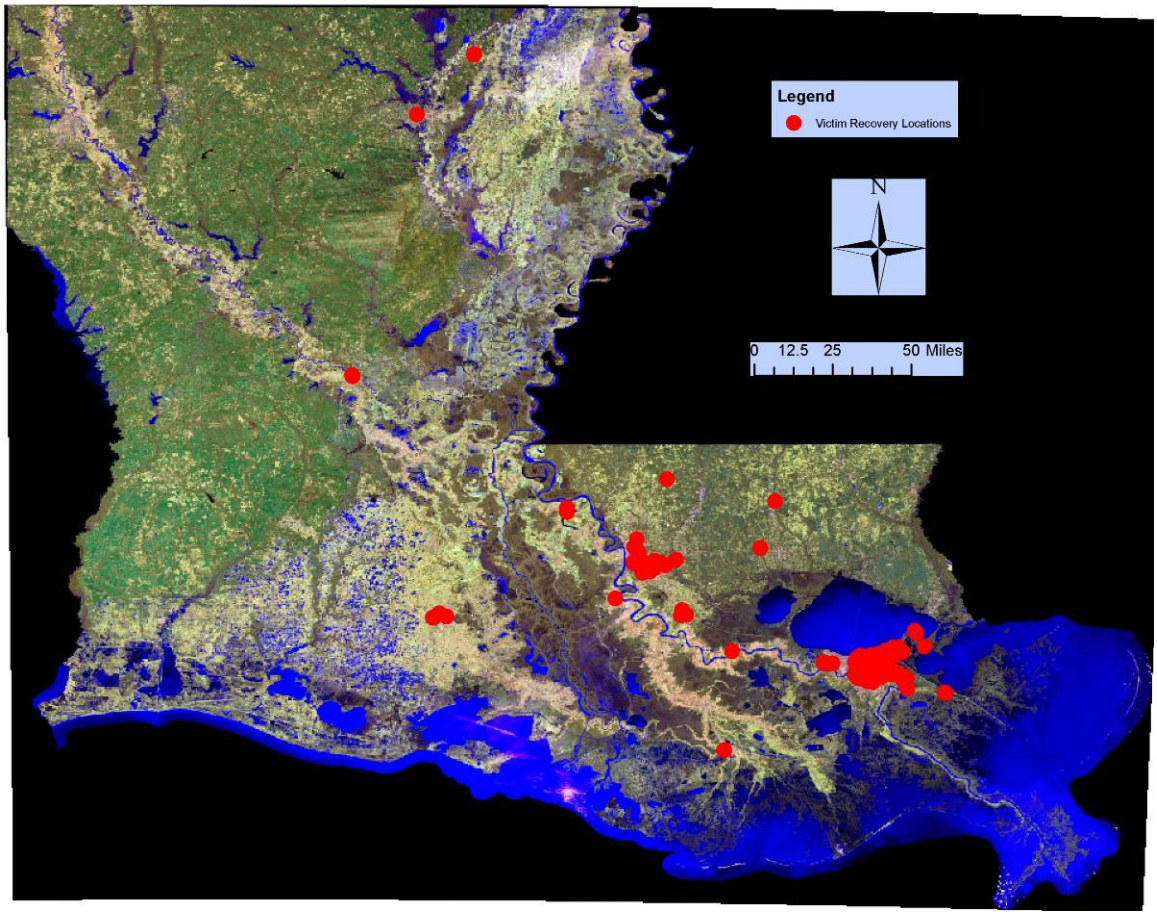

Figure A3.1. Locations of victims recovered in Louisiana.

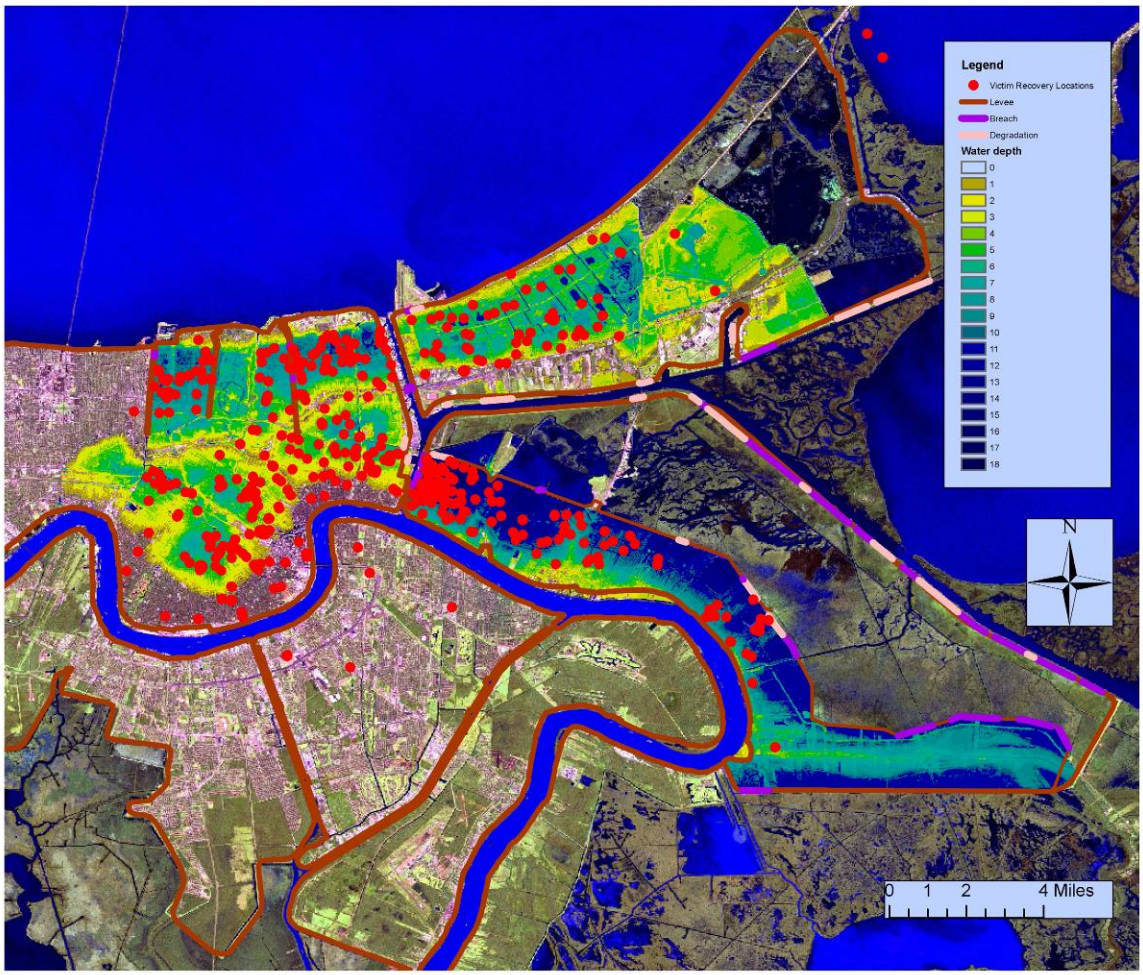

Figure A3.2 Geocoded victim recovery locations within the Greater New Orleans metropolitan area. Note that victims recovered from St. Rita's Nursing Home are not represented here.

Table A3.4 Fatality model forecast from September 8, 2005 for the Katrina flooding event.

Potential fatalities by parish:

	Min	Max
Jefferson:	0	9
Orleans:	1507	3145
St. Bernard:	1	49
St. Tammany:	3	30
Plaquemines:	106	172
Total:	**1618**	**3504**

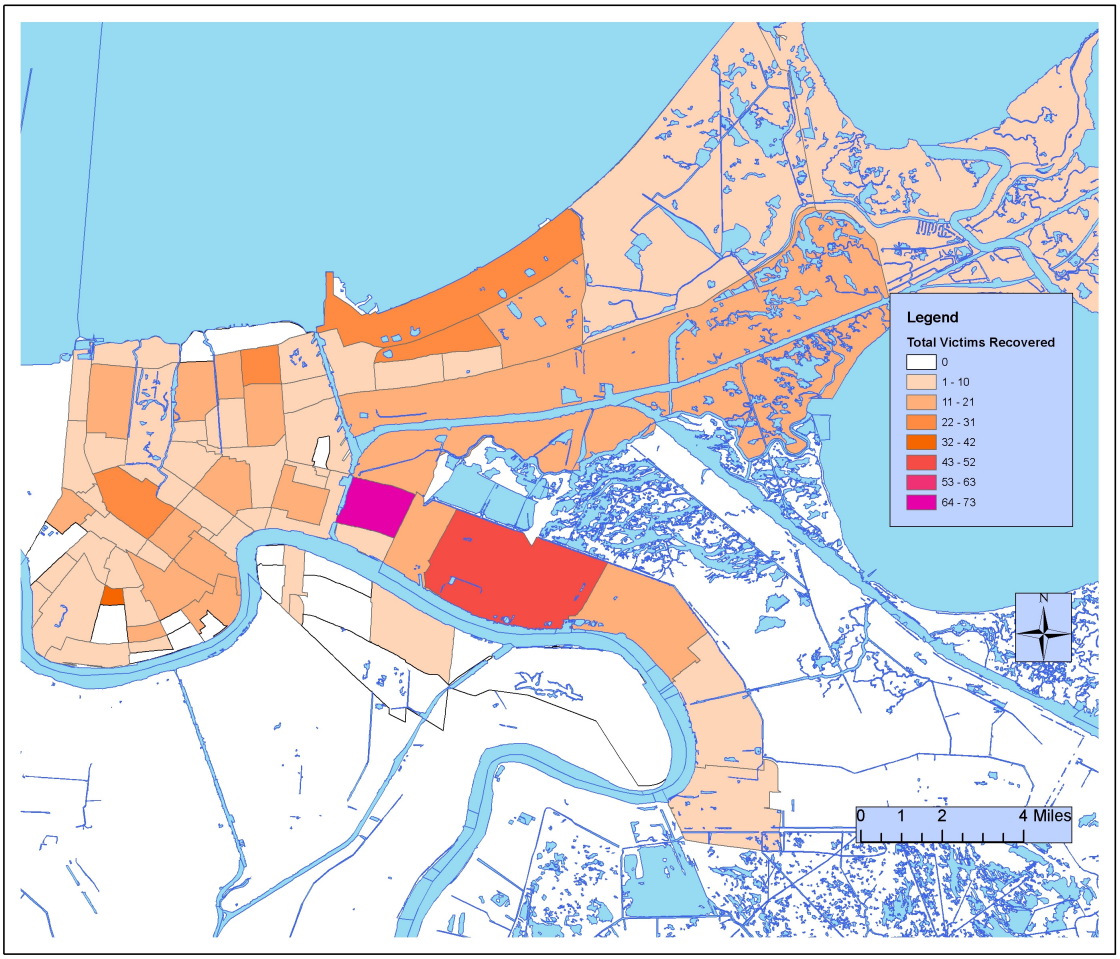

Figure A3.3 Location of recovered victims in Orleans Parish by neighborhood. Neighborhood shapefile provided by GNO Community Data Center.

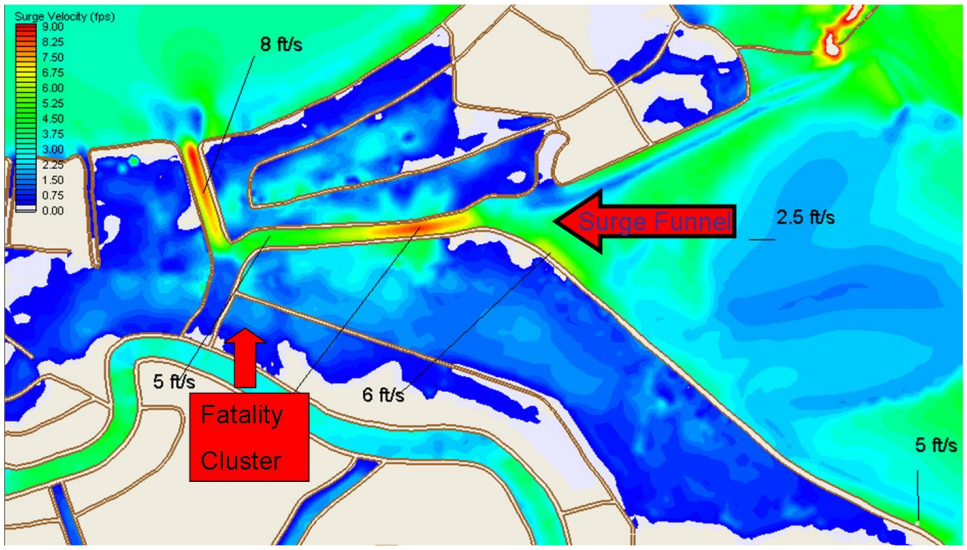

Figure A3.4 The velocity profile of the Katrina surge event demonstrates the high degree of spatial correlation between the fatality cluster and the end of the surge funnel in the Lower 9th Ward. Note that the expectation of low flow velocity in southern section of the Industrial Canal reflects the assumption that the levees held. When the Industrial Levee gave way in the Lower 9th Ward, water flowed into this neighborhood at 6 – 7 ft per sec.

APPENDIX 4. CLOSING NEW ORLEANS' BACK DOOR:
A PLAN TO REDUCE CONSEQUENCES OF THE MISSISSIPPI RIVER GULF OUTLET AND IMPROVE FLOOD PROTECTION

The Mississippi River Gulf Outlet (MRGO) is a classic example of American water resources development policy gone awry. A small number of businesses have no doubt benefited since 1963 from the generosity of federal and state taxpayers who paid for it initially and continue to pay every few years to reopen it when storms silt in the Breton Sound reach (Figure A4.1). Maintenance costs are estimated at nearly $20 million per year. But the citizens of New Orleans and St. Bernard have now paid an unforgettable and unforgivable price for New Orleans' "surge highway" that cannot be ignored the way it was after Hurricane Betsey (1965). While the hexed MRGO channel has been used by remarkably few ocean-going ships over its 40+ year lifetime, it has caused destruction of nearly 11,000 acres of wetlands and has dramatically degraded the ecology of the Lake Pontchartrain estuary (van Heerden and Bryan 2006).

It has been known at least since Hurricane Betsy that the convergence of levee systems following the MRGO and Gulf Intracoastal Waterway (GIWW dredged in 1944) created a 'funnel' geometry with the mouth open to Lake Borgne on the east. This geometry favors the building up of surges generated by cyclones tracking over the city or just to the east (Figure A4.1). Hurricane-generated storm surges arise from the interaction of storm-forced water flows with the geometry of the receiving coastal landscape. Like a tsunami, the elevation of the surge is trivial in deep water but rises up against any barrier, whether a barrier island, beach, floodwall or levee, particularly if there is no ready avenue for the water to escape. In coastal Louisiana, the hurricane protection and river levees are the highest features in the landscape, so this is where the surge rises to its greatest heights. While the throat of the funnel is a large navigation channel (40 by 500+ ft), it is small relative to the gape (nearly 10 miles) so that water pushed into the jaws tends to pile up (Figure A4.1).

A consultant's report surfaced earlier this year that purported to address the effect of the MRGO on storm surge (URS 2006). It was revised and repackaged in part as an appendix of the second report issued by the U.S. Army Corps of Engineers Interagency Performance Evaluation Team (IPET 2006). IPET (2006) cleared up some of the confusion left by the consultant's report but still concludes that the influence of the MRGO on regional surge elevations is very small, generally less than a foot, for a range of storms including Katrina. These

results are based on output generated by the ADCIRC hydrodynamic model also used by the LSU Hurricane Center (Mashriqui and Kemp 2006). The results are a useful contribution but do not answer the most important questions and have been widely misinterpreted.

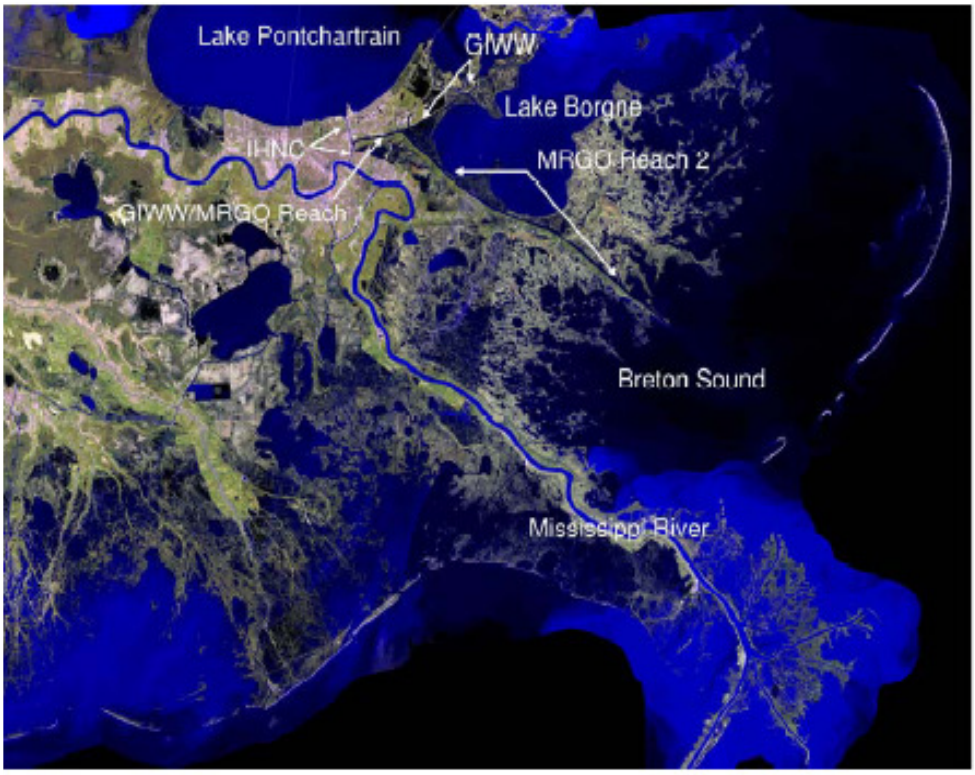

Figure A4.1. Funnel area east of New Orleans showing MRGO Reaches 1 and 2 adapted from IPET (2006)

It is very important to understand that the consultant's report does not address the entirety of the MRGO channel, but only that portion that extends southeast of the throat section (Reach 2, Figure A4.1). The consultant raised the ADCIRC computational nodes that previously were in Reach 2 of the MRGO channel to marsh elevation to simulate a filling of that part of the channel. The dimensions of the throat reach, called Reach 1, is acknowledged in the IPET report to be the "critical reach" with regard to the propagation of the surge into central New Orleans than Reach 2. When we modeled the Katrina surge, we located the critical reaches in the system by looking at the zones of peak velocity (Figure A4.2). One of these peak velocity zones occurs in Reach 1, showing that surge propagation would be most sensitive to changes of cross-section in this region.

But the consultant left Reach 1 unchanged in the study rather than returned to the dimensions of the original 12 ft deep barge channel, the pre-MRGO condition. This would have resulted in a 10 fold reduction in the cross-section for this reach. So they proved that the non-critical Reach 2 was, in fact, non-critical. Results were presented only in terms of the difference in maximum surge elevation with and without the Reach 2 channel segment. So, the lesson is that if you ask the wrong questions, you are likely to get the wrong answers. Let's look a little closer at the dynamics.

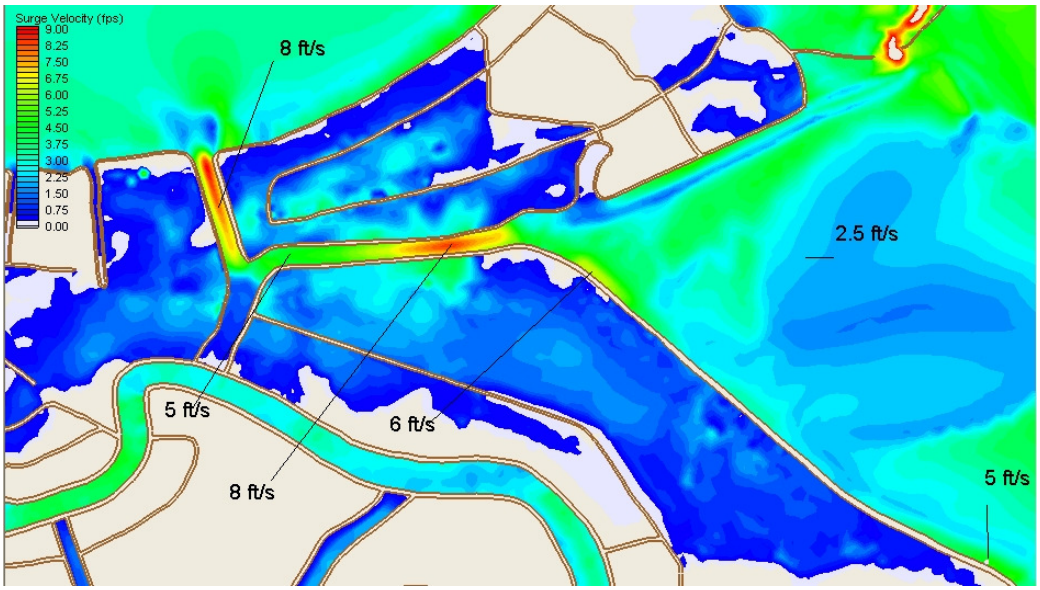

Figure A4.2. Peak flow velocities in the funnel in Katrina simulation. The red regions are zones of highest velocity and show that MRGO Reach 1 and the northern reach of the IHNC are 'critical' and would be the places where a reduction in cross-section would have the most effect on surge propagation.

A storm surge, unlike a river flood which lasts for days or months, is an event that is measured in hours (Figure A4.3). The storm will typically build up a surge sequentially against different landscape features as it approaches and moves through (Figure A4.4). Similarly, the cyclonic winds at any single coastal location may initially be alongshore,

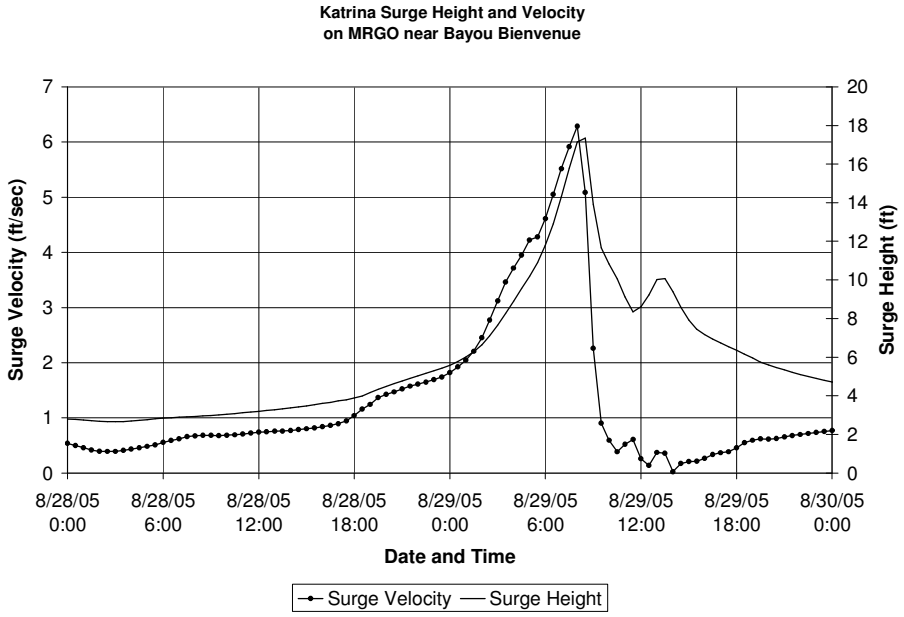

Figure A4.3. Surge elevation and current velocity in the MRGO channel in Katrina ADCIRC simulation (adapted from Mashriqui and Kemp 2006).

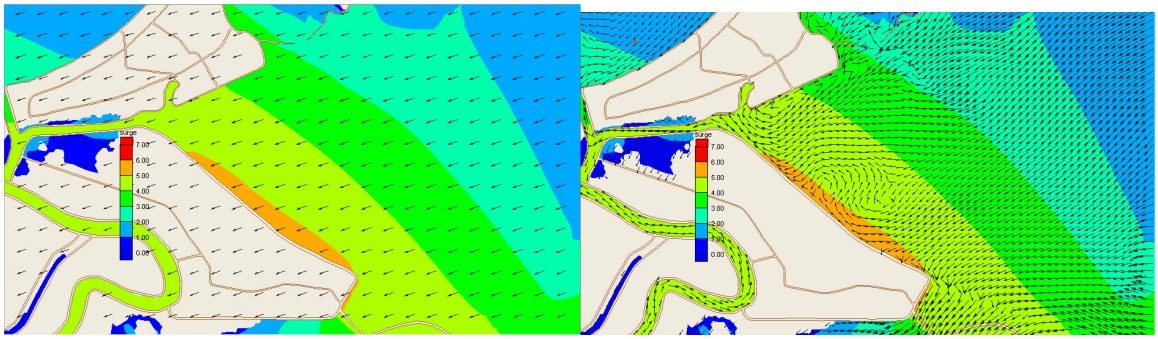

Figure A4.4. ADCIRC Katrina snapshot showing surge piling up on the MRGO levee in both panels, with winds on the left and current direction on the right.

then onshore, and finally offshore. The coastal currents generated by the winds do not generally flow exactly in the direction of the winds but are strongly influenced by features like channels and levees even when these features are largely or completely submerged. In the MRGO, for example, while the wind may blow across the channel, the highest flow velocity is always along the axis of the channel. Whenever we are looking at the effects of a storm surge on levees, we have to consider not only how high the water got, but the duration of exposure to high water, and whether the high water made it possible for waves to reach the levee face. The hurricane time dimension means that if an efficient channel is absent, it will take longer for the surge to reach a given height. A fast moving storm may move on before that height is attained or, for a slower storm, the duration of high water is reduced. In both cases, the stress on the levees and floodwalls is also diminished. For a small, fast moving storm, distant landscape

features like offshore islands, low ridges and wetlands may block the propagation of surge inland. These features will have less effect on a large or slow-moving storm, where levees must take more of the brunt of the impact.

The funnel includes a large lake, Lake Borgne, some remnant wetlands west of the lake, and two artificial canals, the GIWW and the MRGO, with the MRGO larger in cross-section by a factor of ten (Figure A4.1). During the storm approach sequence, water flows into the funnel from the east and finds the most efficient route to the drain at the throat (Figure A4.4). Water can flow relatively slowly over flooded wetlands that are shallow and offer a relatively high resistance, or it can move more quickly in a deep channel that offers far less resistance (Figure A4.5). While water could enter MRGO Reach 1 from anywhere in the funnel, it is apparent that the MRGO channel is the main source, resulting in a return flow to the east following the northern limb of the funnel and the smaller GIWW channel. MRGO Reach 1, the throat of the funnel, has an appetite for water, a conveyance capacity, that will be satisfied either by water entering from the MRGO or GIWW, or, alternatively, from across the wetlands. So we can see how the consultant's exercise of closing off MRGO Reach 2 merely changes the route that water takes to get into the throat, rather than the volume. To change the surge height in the IHNC, we will have to reduce the cross-section and thereby the conveyance of MRGO

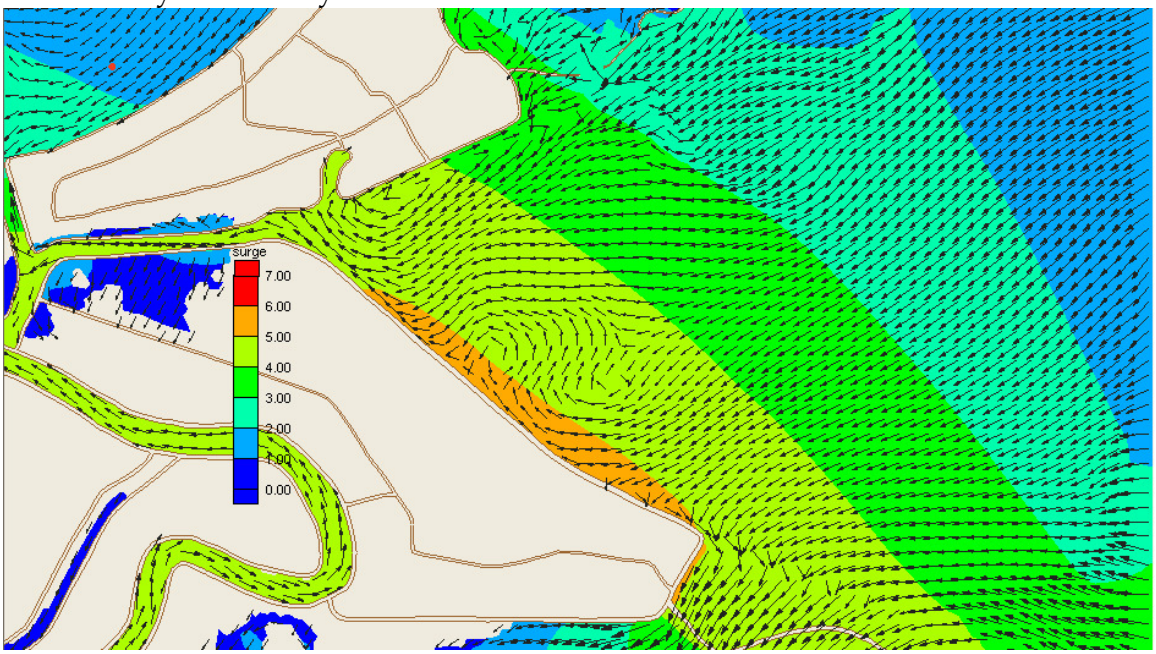

Figure A4.5. ADCIRC simulation of patterns of water circulation at peak surge elevation showing the importance of the MRGO channel along the south limb of the funnel in feeding the throat.

reach 1. During Katrina, once the surge in the funnel reached higher than +10 ft, floodwalls along the GIWW and IHNC began to slide and fall over, while waves began to open breaches in the earthen levees adjacent to the MRGO. Then water began to flow through these openings. Because the two artificial channels follow the alignment of the hurricane protection system, they are positioned to provide a very efficient and essentially infinite supply of water to the breaches. At one time, we calculate that flow through 5 to 6 miles of breaches on the MRGO levee reached as high as 2 million cfs, more than twice the amount of water allowed past New Orleans on the Mississippi during the highest floods.

The MRGO, one of the largest dredging projects in the world at the time it was built, is a central feature that defines and contributes to the hazard of the funnel. This project determined the dimensions of the critical reach that constitutes the throat of the funnel, referred to in IPET Report 2 as Reach 1. This channel is at least three times deeper than any natural tidal bayous in this part of the Pontchartrain estuary and 10 times greater in cross-section than the pre-existing GIWW barge channel. It is understood that Reach 1 permits significant influx of saltwater into Lake Pontchartrain through the northern arm of the Inner Harbor Navigation Canal (IHNC). As happened during Katrina and would be true for many critical storm tracks, the surge from Lake Borgne was higher and earlier than that in Lake Pontchartrain and was communicated into the IHNC via Reach 1. Without the Reach 1 connector, the IHNC floodwalls would have seen a surge of 11 to 12 ft coming from Lake Pontchartrain, similar to what was experienced by the walls along the 17th St., Orleans and London Ave. drainage canals. The Lake Borgne surge added an additional 3 ft. The flow from the funnel also prolonged the period of high water that had to be resisted by the IHNC floodwalls and added to the dynamic impact of moving water on these structures (Figure A4.6). They failed catastrophically in many places.

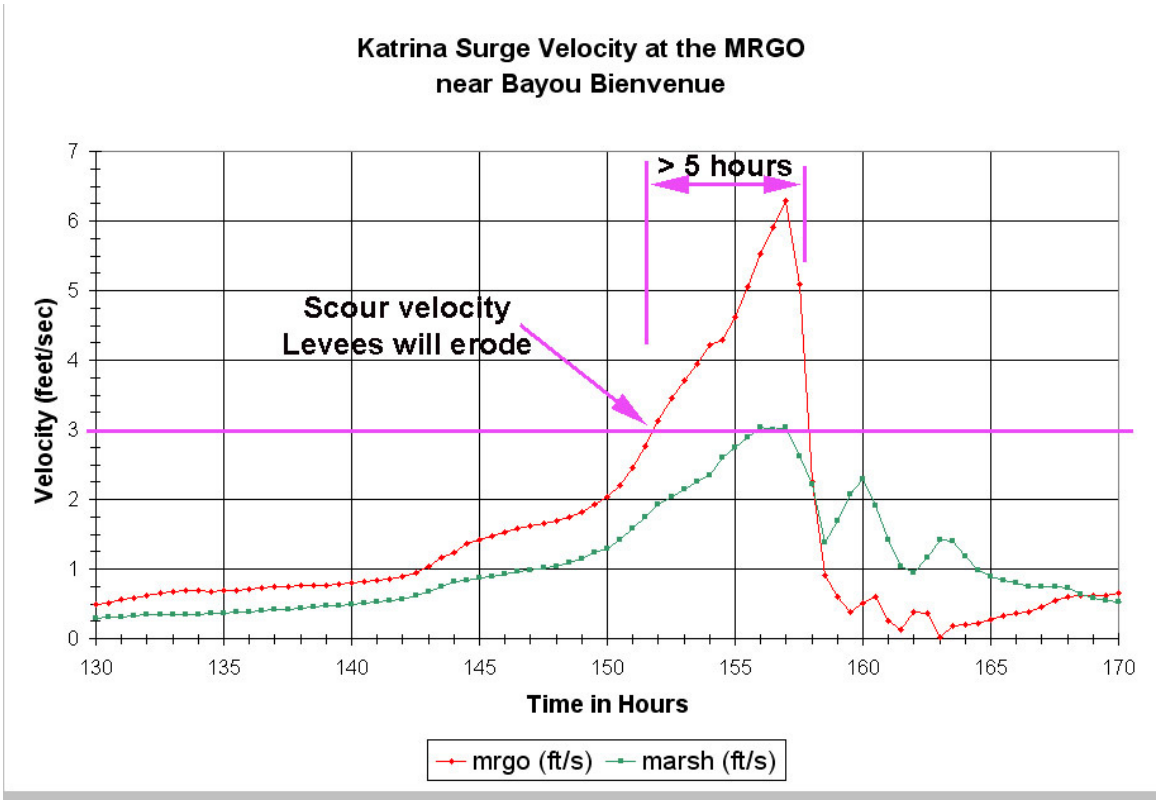

Figure A4.6. History of current velocity at adjacent points in the MRGO channel and nearby marsh in Katrina simulation.

So there is a very strong storm surge based argument to close or restrict MRGO Reach 1. A closure in Reach 1 could also manage the salinity intrusion into Lake Pontchartrain. What about MRGO Reach 2? The main rationale for closing MRGO Reach 2 has, in the past, come from the environmental community that has watched the channel widen over time as its unstable banks gradually fail and fall into the maintained channel cut. It appears, however, that there may also be a problem in putting such a large channel in such close proximity to the main hurricane protection levee. We have discussed the way the channel can supply water to breaches, but what does a channel a thousand feet wide at the toe of the levee do for waves? We have identified much evidence that waves attacking the face of the MRGO levee led to its breaching early enough (10 to 12 ft elevation) to flood the populated area of St. Bernard after filling an intervening 31,000 acre wetlands (Figure A4.6). Similar levee reaches survived if they were protected from waves by a wide enough wetland apron, enough to break the waves before they reached the levee face. Past history demonstrates that the Corps is unable to stop the enlargement of the MRGO channel, whether it takes out wetlands on the north side or eats into the toe of the rebuilt and still unarmored levee. It is just

poor practice to leave that vital levee so exposed to lateral erosion from an adjacent channel without any real value as a navigation channel.

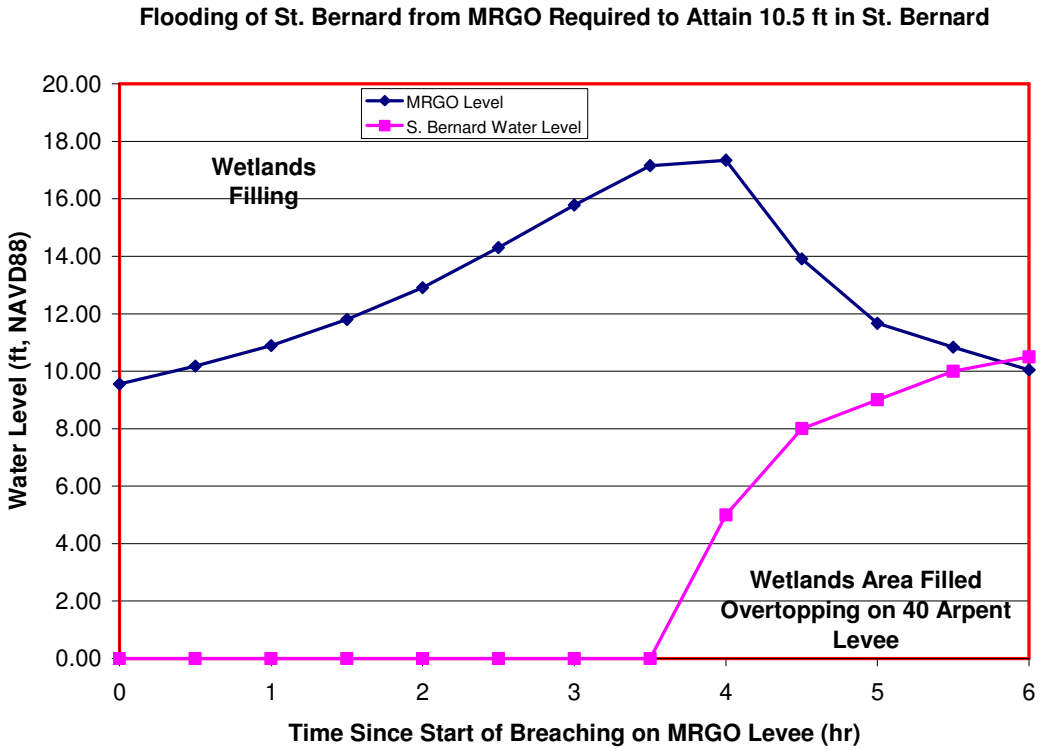

Figure A4.7. Comparison of water level in the MRGO in a Katrina simulation with that predicted by St. Bernard flooding model in the protected communities. Note that buffer wetlands fills for 3.5 hours before flooding of the communities begins at about 0800. Flooding is complete at 1030.

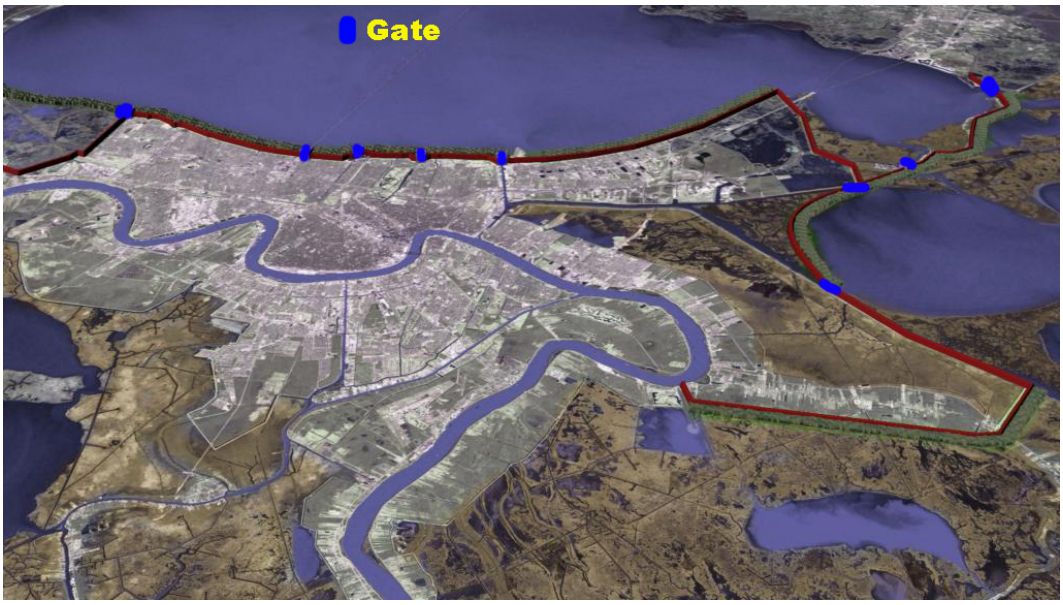

Figure A4.8. One concept for breaking up the geometry of the funnel and reducing surge.

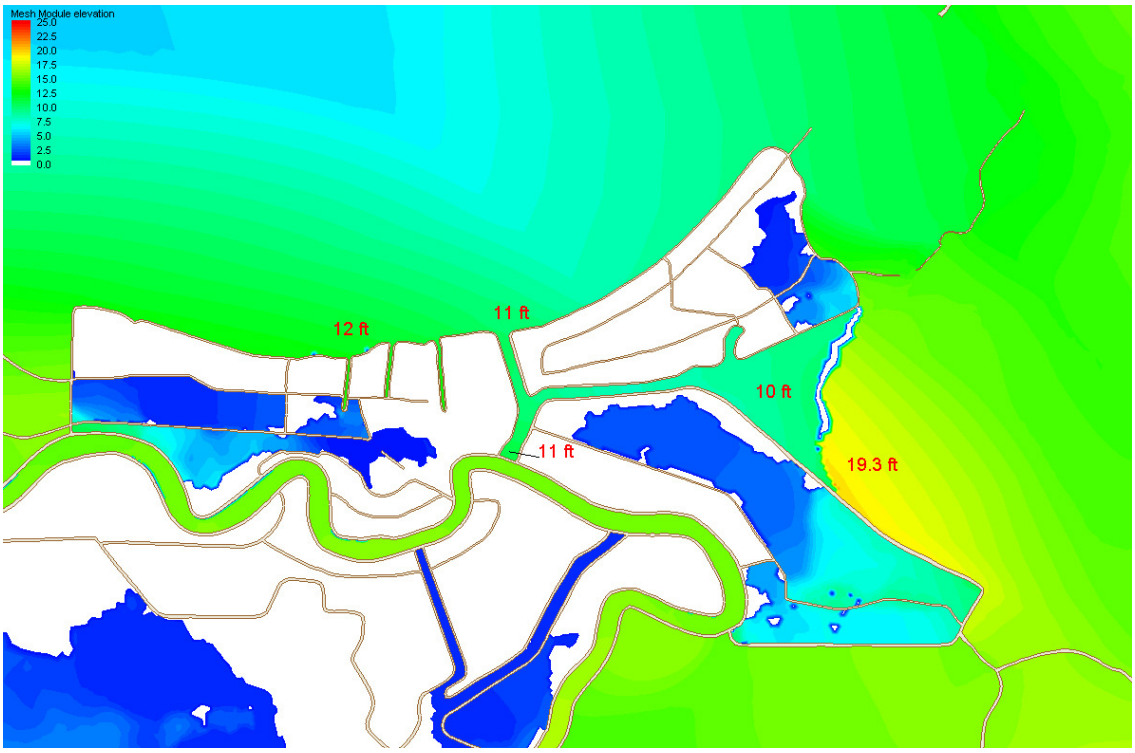

Figure A4.9. Effect of a curved levee on surge along the MRGO and GIWW levees. Note that peak surge in the southeastern part of the MRGO levee is raised by about 2 ft, though it is greatly reduced elsewhere in a Katrina ADCIRC simulation.

One alternative levee alignment under consideration is one in which the main hurricane protection levee is pushed to the east along the western margin of Lake Borgne (Figure A4.8). This would result in a greater rise of surge along a shorter part of the MRGO levee, as we have cut off the single route for drainage (Figure A4.9). Several opportunities to restore wetlands on a large scale exist if the existing MRGO channel is filled in partially or wholly, or in segments.

APPENDIX 5. A Conceptual Plan for the Long-term Protection of Coastal Louisiana from the Ravages of storm Surges

A5.1. Big-picture Planning

Any big-picture surge protection system for Louisiana this state must include major levees and flood gates in conjunction with creating wetlands and barrier islands. Hard structures should only be built where they can be protected by an existing "platform" of wetlands, no matter how fragile, and where there's a reasonable supply of good soil building material for levees. The wetland platforms seaward of the levee and floodgates should be the target sites for future wetlands creation projects, with barrier islands seaward of the expanding wetlands base. Levees—wetlands—barrier islands: This progression assures the survival of the estuarine bays necessary to keep the commercial and recreation fishing industries alive, as well as to supply the breadth of natural habitats that make up this unique ecosystem.

To date no plan has been forthcoming from any agency so we summarize that presented by van Heerden and Bryan (2006) below:-

• Build a flood control structure with multiple gates in the Rigolets along the I-10 Bridge similar to the Dutch Eastern Schelde barrier. At least one shipping lock at the highest part of the bridge over the navigation/tidal channel will be needed. If designed in conjunction with the use of numerical tide models the structure would address the environmental issues that helped stall the idea forty years ago. Such a structure would mean that we would *not* have to raise any of the lakefront levee systems in New Orleans. All we would need to do is "armor" them. Examples of such armor include a shield of tightly fitted nine inch-thick concrete blocks with a loose gravel covering, or a layer of asphalt or bitumen mix as protection against surface erosion. These floodgates in the Rigolets would also protect the north shore of Lake Pontchartrain and communities in huge areas of St. Tammany, Tangipahoa, and Livingston parishes as well as parts of East Baton Rouge Parish (Country Club of Louisiana).

• The pump stations must be moved to the head of the 17th Street, London Avenue, and Orleans Avenue canals, blocking any storm surge from the lake from entering the canals. Obviously, the lakefront gates presently being constructed create a problem if we have a storm with high surge and lots of rainfall. With the gates closed to protect against the surge, the existing pumps could not operate should there be flooding from rainfall. Pumps are required along the lakefront, fully automated, computer-driven, and with standby

generators to make them immune to power failures. No one should have to stay behind to operate them.

• A gated structure with either a large lock or a surge barrier plus a lock needs to be built where the Ted Hickey Bridge crosses the Industrial Canal to stop surge flowing to and from lake Pontchartrain and lake Borgne.

• Eastward from the Rigolets Barrier system the levees from Orleans East to the Intracoastal Waterway have to be substantially raised and made into major levees. (Likewise, we must have a major levee west of the Rigolets to protect Slidell and possibly parts of coastal Mississippi.) East of New Orleans we need a butterfly-gate type of flood control structure in the area of the funnel. It could be a copycat version of the Dutch structure at Maeslandt. We need substantial levees extending from this butterfly

gate and linking with the levees along the Mississippi River that protect from river floods. This could be a rebuilt version of the 40 Arpent Canal levee. All levees should have armoring that saves them from both wave attack and erosion should they be overtopped.

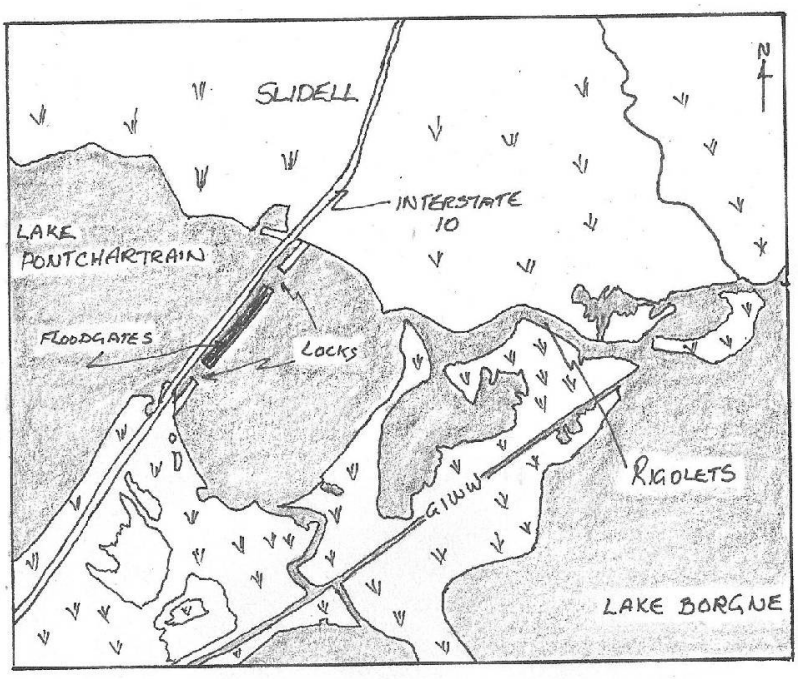

Figure A5.1. Location of surge floodgates and twin lock system proposed for the Rigolets at the I-10 Bridge. This would protect not only New Orleans but all the parishes bounding lakes Pontchartrain and Maurepas (from van Heerden and Bryan 2006)

• From the Mississippi River westward we need to build one giant levee that basically follows a bulging curve seaward, taking in Houma and some of its surrounding communities (some smaller curves may be necessary to incorporate

some communities). With careful location, this levee, about eighty-four miles long, can balance the protection of the communities "inside" and wetlands fronting the system on the "outside." This inland levee would include numerous gated structures at key navigation channels and where river/sediment/freshwater diversions will be built. This levee would then tie into the East Guide Levee of the Atchafalaya River at a point close to Morgan City. Thereafter the levee can be extended from the West Guide Levee of the Atchafalaya River all the way to Texas, with navigation gates at the required locations. This giant levee needs to be on the seaward flank of a large navigation channel, parts of which would include the existing Intracoastal Waterway. This channel would allow access for heavy equipment and barges with soil, rock, and other levee-building materials. To some extent this new levee would replace the network of levees now in place in this area south and west of the Mississippi River. The existing network creates some funnel effect problems of its own.

• This whole system—levee with navigation/sediment distribution gates—must be accompanied by legislation that stops development in the wetlands anywhere in the newly protected areas. Otherwise our current river flooding problems will be exacerbated, because these gates, and especially those at the entrance of Lake Pontchartrain, will slow down drainage of water out of the lake after major rainfall floods. Likewise, there needs to be a moratorium on the mining of cypress forests for mulch anywhere in coastal Louisiana. Cypress swamps are some of our best defense against storm surge. Many communities are going to be outside this new levee system. Some retreat has to take place. Communities outside the new system could be given some level of flood protection, but protection that costs more than the value of the protected infrastructure is, clearly, problematic, and will be hard to sell to the federal government. It should also be required that all new construction within the new barrier be elevated a given number of feet as determined by the design specifics of the whole system. This would provide an extra margin of protection in the event of some levee overtopping, or to reduce flood damage should there be a rainfall flood that exceeds the capacity of the pumps.

In order to protect the levee system, and as a hedge against a rising sea level and climate change, we must concentrate on building and restoring wetlands seaward of the major levee, using the sediment resources of the Atchafalaya and Mississippi rivers. Even with the major levee system, these wetlands are still the outer defense and must be maintained for this purpose, as well as for their commercial, recreational, and environmental uses. Natural systems and some developed areas within the core should also be maintained, in order to accommodate expected overtopping and to avoid the degradation of water quality that the Dutch have experienced. Seaward of these wetlands we need to

build barrier island chains, close in to the existing wetlands, using high-quality sand from offshore.

The major elements necessary for large-scale wetland creation include:

• Below New Orleans, divert the Mississippi River into the Breton and Chandeleur Sounds, 32 miles upstream from the present main outlet (Figure A5.2). The Breton and Chandeleur sounds are dominated by shallow bays that have modest subsidence rates and are therefore excellent potential recipients of river sediment. This new diversion would create five thousand new acres of marshland every year in eastern Plaquemines and St. Bernard parishes. In twenty years we would have 140 square miles of new wetlands to buffer storm surges. The plan would mean the loss of the bird-foot delta. This dying delta could be reworked into new barrier islands that would coalesce with the Breton-Chandeleur islands to the east and the shoals and islands of western Plaquemines Parish, thus creating an almost unbroken barrier island arc south and east of New Orleans and its suburbs. The plan would also eventually require, in all likelihood, a set of locks and other control structures at the new outlet. Some oyster beds would have to be relocated.

• Reconnect Bayou Lafourche to the main channel of the Mississippi River fifteen to twenty miles to the east of its junction with the Mississippi and divert 10 percent to 12 percent of the total flow down this channel, which would feed the marshlands all along its course, bring freshwater to 8 percent of the state's population, and create jobs in the fisheries. Dredging the bayou would be necessary, along with extensive plumbing that would include small diversions and siphons.

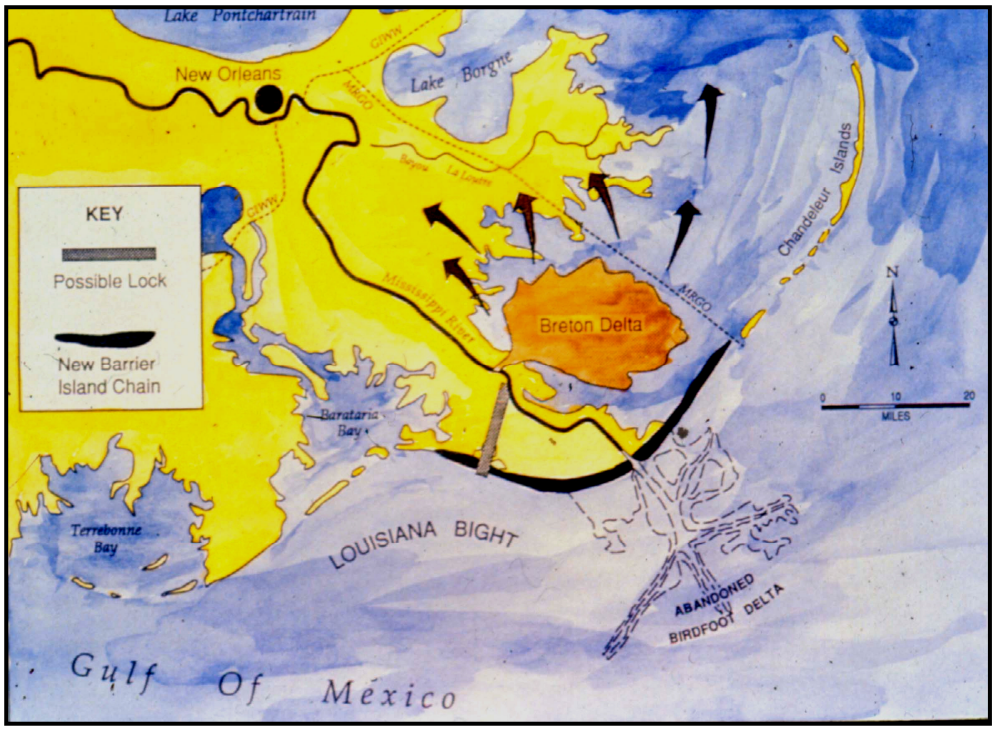

Figure A5.2. Conceptual view of proposed Breton Delta

• Increase the flow down the Atchafalaya from 30 percent to 40 percent, while using plumbing to manage the extra water and sediment flowing into the Terrebonne, Atchafalaya, and Teche-Vermilion basins. The east-west–oriented Intracoastal

Waterway is an excellent conduit to move sediment laden water miles laterally; the boat traffic's propellers keep the sediment in suspension. The canal dredged behind the barrier levee where it did not follow the Intracoastal Waterway could act as a similar conduit. Throughout the system, large- and small-scale diversions and siphons could help mimic the natural annual floods. The system needs to be as "leaky" as possible.

Past experience in the form of the Wax Lake Outlet delta have shown that fresh water diversions do work. This canal, between the Lower Atchafalaya River and the Gulf of Mexico, was cut in 1948 as a relief valve for floodwaters that would otherwise hit Morgan City a few miles to the east. The channel was cut through wetlands and a series of small lakes with no confining levees for the last ten miles of its length. Since the diversion offered a shorter course to the Gulf than the actual river course, this channel maintained itself; that is, it required no dredging and even started to capture more water from the river. Discharge from the Wax Lake Outlet has averaged about 10 percent of the total flow of the whole Mississippi system. Seaward of the outlet we have a great evolving natural delta

with extremely healthy natural levee and marsh environments, a delta that expands with every flood with no help from man.

• The fourth initiative is to restore and/or build new barrier islands. As they disappear, wave energy increases as do storm surges, and saltwater encroachment. Much of the oil-and-gas infrastructure is old and not engineered for the resulting quasi open-ocean conditions. Ship Shoal, a major sand resource eleven miles south of Isles Dernieres, could be mined for the purpose. We know that a healthy barrier island with a good sand beach berm—and wide wetlands behind the berm—is a long-lived feature. Adding sediment to the islands would have to be undertaken from time to time because there would probably be losses in every big storm, but as the islands are being eroded, they are in turn reducing the storm surge to a measurable extent. Not restoring them dooms all remaining wetlands to a far more rapid death. Additionally, the barrier islands also form the outer edges of the giant estuaries that are the basic framework supporting the very rich fisheries in Louisiana. Lose these and the fishery collapses.

The core protected area in southeastern Louisiana would *not* have to be evacuated for a Cat 5 storm (Figure A5.3). Other areas outside this core would have a higher level of protection than exists today, but to a lesser degree than the core, and they would sometimes require evacuation. This fact can't be allowed to compromise the commitment to protect the core. So this plan is attractive because it proposes a higher level of flood protection for the principal populated areas than any previously proposed. If you never have to evacuate, you can then support a level of infrastructure investment that has not been possible in the past. Restructuring the coastal landscape will mean dislocations and disruptions of some traditional fisheries. Resources will be lost while new ones will be gained. Dislocations should be kept to a minimum, and the planning effort must incorporate compensation for those who lose land or livelihood.

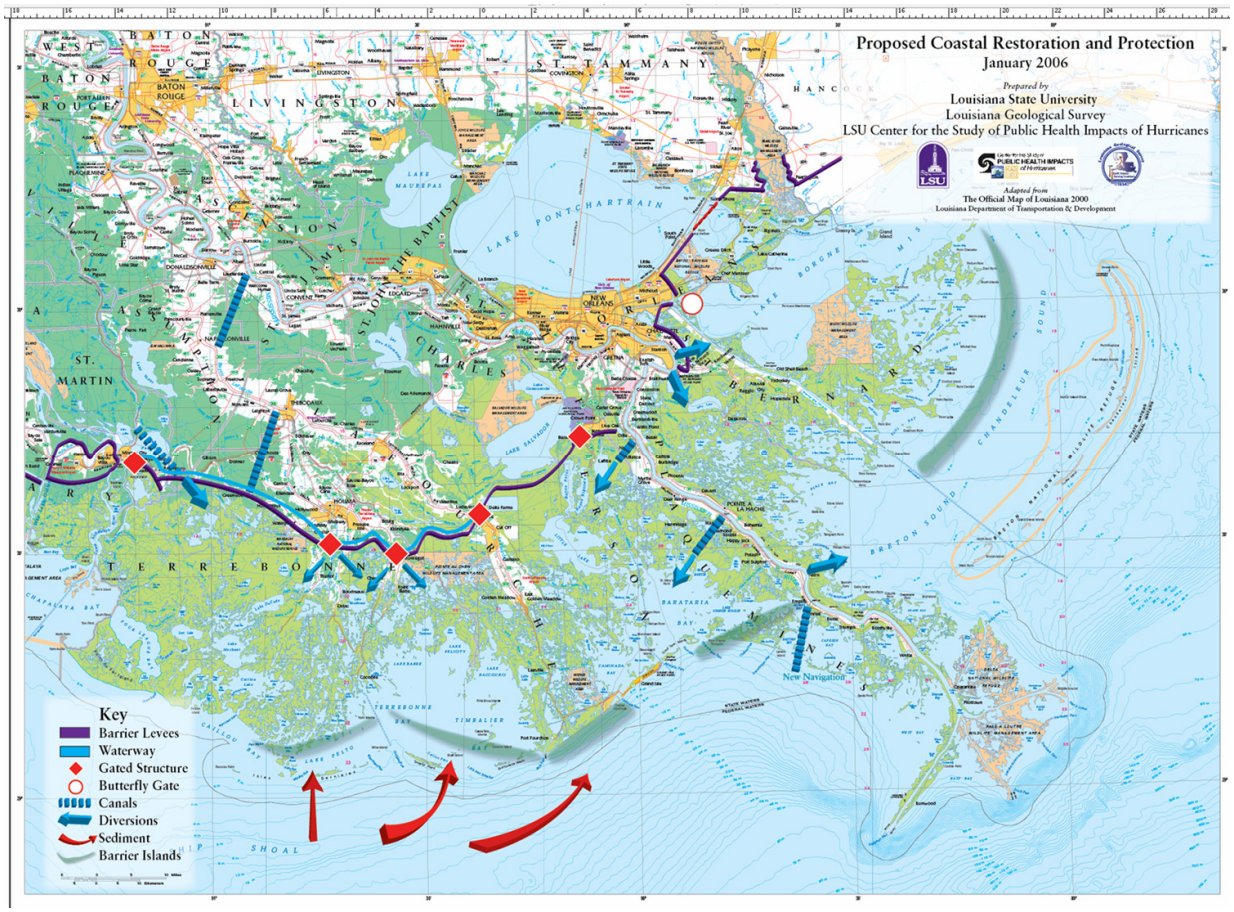

Figure A5.3 Conceptual Plan for Long-term Coastal restoration and Barrier Levees for Surge protection

APPENDIX 6. REFLECTIONS ON THE DRAFT FINAL USACE IPET REPORT TITLED 'PERFORMANCE EVALUATION OF THE NEW ORLEANS AND SOUTHEAST LOUISIANA HURRICANE PROTECTION SYSTEM'

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This is a very important piece of work. It provides extensive insights into the 'engineering mechanics' that lead to the most catastrophic failure of a civil engineered system in the history of the United States. These reflections concentrate on the findings contained in Volume I - Executive Summary and Overview.

Overarching Findings
Yes, the hurricane flood defense 'system' for the greater New Orleans area (NOFDS) was "a system in name only". Even though there had been only one agency in charge of concept development, design, and construction - the Corps of Engineers - it is clear that the NOFDS was not a coherent defense system. It was and continues to be an assembly of disjointed and defective components.

Yes, armoring the back sides and crests of levees and the protected side of floodwalls would have added significant redundancy and reduced breaching. One could wonder why such protection was and has not been provided in the repairs to the levees adjacent to the Mississippi River Gulf Outlet between bayous Dupre and Bienvineu. The original plans for reconstruction of these levees omitted armoring (January 2006). When the plans were revised, only armoring on the front or exposed side was proposed (February 2006). Now, the

need for full armoring of the levees is recognized. It would seem that it has been during the course of the IPET study that recognition has developed in the Corps of Engineers that these levees are exposed coastal defense structures that must be able to sustain the combined effects of waves, currents and surge. Coastal engineers around the world have and do understand the need for such defensive measures for earthen levees. Why is the need for such defensive structures such a recent revelation?

Yes, back-up, fail-safe, and defect - damage tolerant system components are needed. However, it is important to recognize that 'redundancy' in itself does not provide such highly desirable performance characteristics. It takes much more than redundancy; it takes proper configuration (which can include redundancy), excess capacity, ductility, and appropriate correlation. The effects of redundancy can be completely erased by a variety of factors. It is obvious that design guidelines are urgently needed for engineers to develop an adequate understanding of what it takes to develop a defect and damage tolerant components for the NOFDS.

Yes, the storm exceeded the design criteria in some parts of the system. The fact that the design criteria in some parts of the system were exceeded and that these parts of the system performed so poorly indicates that the margins of safety were not sufficient. Engineered systems are expected to perform acceptably even though their design criteria are exceeded; that is the reason for margins or factors of safety. It is clear that even the 'design criteria' (referring to the hurricane conditions) as expressed in the Standard Project Hurricane (SPH) also were insufficient and defective. This technology deficiency started in the 1950s and was clearly recognized by the Corps and others in the 1970s; nothing was done to remedy the deficiencies. This same story plays out again and again throughout the history of development of this system: inappropriate use of technology. Another important example is the elevation datum reference developments that resulted in low protection elevations (2 to 3 feet and more); these were recognized by the Corps of Engineers and decisions were made to accept these low protection elevations. The omission of splash pads behind the original concrete I-walls is a similar case. Design engineers were instructed through the Corps of Engineers design guidelines to design only for the authorized water levels; not for overtopping. Such a strategy results in very 'brittle' components that can not tolerate conditions that exceed the authorized conditions. Even the fundamental factors of safety incorporated into the design guidelines for the levees and floodwalls were unreasonably low. Extensive research into the history

of these factors of safety could provide no substantial justification for their acceptance in this particular application; defense of a major metropolitan city.

Yes, levee sections were constructed with erodible materials. And, these levee sections were constructed according to the Corps construction specifications. These levee sections were approved based on the Corps quality assurance and control procedures and processes (QA/QC). These construction specifications and QA/QC procedures and processes are documented in current Corps of Engineers design and construction guidelines. These construction specifications and QA/QC procedures are being used (sometimes in forms modified to lessen their requirements) in reconstruction of some of the earth levees (e.g. MR-GO levees).

Yes, the floodwalls and supporting levees within the Inner Harbor Navigation Canal and within the drainage canals (London, Orleans, 17th Street) did not perform acceptably. The failure of these elements contributed the vast majority of flood water that devastated the central New Orleans area. They failed before the design water levels occurred. They failed because of a variety of reasons that can be grouped into what are commonly termed
'engineering failures.' But, these engineering failures were not the causes of the breaches. The failures resulted from inability of the Corps of Engineers to properly apply existing technology. Many of the important parts of this technology were developed by the Corps of Engineers and inexplicably the Corps of Engineers was unable to take advantage of its own developments. There are ample signs of rejection of technology and signs that indicate lethal arrogance.

Yes, the majority of the flooding can be attributed to unanticipated and undesirable breaches in the flood protection system. Our work indicates something in excess of 75 to 80 % while the Corps of Engineers work indicates approximately 67%. In some areas, our work indicates as much as 90% (St Bernard Parish). So, there is general agreement that the majority of flooding, damage, death, and suffering can be attributed to breaching that should not have occurred.

Yes, in most cases it is likely that the repaired sections of the hurricane protection system are the strongest parts of the system. We understand that it takes a coherent and integrated system to provide proper protection. Strong pieces embedded within weak pieces does not translate to a reliable system. All parts or components of the NOFDS are suspect until it can be proven beyond a

reasonable doubt that they will perform acceptably. That confirmation has not been provided. A prime example of shifting the problem from one location to others regards the installation of gate closures for the drainage canals. These gate closures when completed will provide protection against hurricane surges entering from Lake Pontachatrain. However, at this time and in the foreseeable future, there is insufficient pumping capacity at the lake front to allow the pump stations located at the heads of the canals to operate at their full capacity to empty the city of rainwater and floodwater. This then shifts the problem to the levees and floodwalls that line these canals and these have already proven marginal or insufficient for significant water levels in the canals. The question then is how much water can be allowed to build-up in the canals? Depending on the factors of safety that would be required, the answers seem to range from 5 feet to 8 or 9 feet. Will the pumps be able to keep up? Will the pumps operate as intended? What will happen if the pumps can not keep up with the water pumped into the canals?

Synopsis of Principal Findings

Geodetic vertical and water level datum

Yes, subsidence was not properly accounted for. Yes, revisions in reference datum were not recognized. Yes, there were errors made in references of datum to water level. Multiple errors accumulated over a very long period of time (40+ years). Corps of Engineers documents do indicate that the potential errors were detected and recognized; however, they were not corrected. The decision was made to construct the works to the elevation datum referenced at the time of the original authorization of the works (1965). The accumulation of these errors lead to walls and levees being constructed 2 to 3 feet or more lower than intended; resulting in substantial compromises in the reliability of the NOFDS.

Hurricane Protection System

The first sentence "There was no evidence of government or contractor negligence or malfeasance" seems to be misplaced in this document. It has not been documented by the Corps of Engineers that such assessments or evaluations were within the scope of this study. No corroboration or substantiation of this statement has been provided. This statement appears to be very self-serving and detracts from the quality and credibility of the engineering forensic studies incorporated into this important study. Based on the evidence (written, verbal) gathered during the course of the ILIT study, there was and is evidence of government negligence and malfeasance. Negligence is defined as a failure to exercise the care that a prudent person would. Malfeasance is defined

as wrongful conduct. The documented background for development of this flood protection system during a period of more than 40 years indicates many instances of both negligence and malfeasance. We received uncorroborated written reports of instances in which inspectors were told to disregard construction specifications and construction quality control reports falsified. Statements included in the IPET report such as "generally built", "consistent with local practice", and "with exceptions noted below" do not relive the Corps of Engineers of allegations of negligence and malfeasance. Such determinations must be left to the courts and our legal processes.

Yes, the original design criteria represented in the Standard Project Hurricane were not and are not appropriate.

Yes, the methods used to evaluate the soil shear strength, factors of safety, and levee - floodwall stability were not and are not appropriate.

Yes, the incomplete sections (e.g. north of the Orleans pump station and south of the 17th Street pump station), missing railway floodgates, trees at levee toes, in-ground construction at levee toes, were all contributors or 'enablers' in the disastrous flooding. The trees required long periods of time to grow to their size (it is noteworthy that trees blown over by the hurricane winds were observed at the centers of each of the drainage canal breaches and were located at the critical toes of the levees). Multiple required inspections of these components in the flood defense system indicated that 'all was well.' How did these infractions in the requirements go unrecognized, unreported, and unresolved by so many people for such a long period of time?

Storm

Yes, the water levels developed in some parts of the NOFDS did exceed the design criteria. But, as noted, it was known since the 1960's that the SPH conditions that defined the criteria were not representative of those that should have been used to design the NOFDS. The degree of exceedance of the conditions varied from place to place in the NOFDS. The degree of exceedance is indicated by results from calibrated analytical models and the results from these models differ. The presence of dynamic forces is also to be expected when surge current velocities and wave forces are present. The contributions of these forces are also uncertain.

Yes, overtopping by surge and waves on the fronts, crests, and backs of the levees all lead to high potential for scour and erosion. These erosive stresses

varied from place to place and had varying effects depending on the orientation of the stresses relative to the levees, the soil characteristics, the presence of natural defenses (wetlands, trees, grass) and the presence of engineered defenses (that sometimes had exacerbating effects). Our work indicates that there were many significant cases in which the levees that failed due to erosion were caused by erosion of the front face; this finding contradicts the IPET finding that "the levees that failed due to erosion determined that all were caused by erosion of the crest and back face." Our work is continuing to examine the cases where erosion and other hydrodynamic and hydraulic phenomenon during the surge build-up brought severe wave action and current action against the front faces of the levees, floodwalls, and flood control structures. Evidence in some important cases indicates breaching prior to overtopping.

Yes, the southeast trending leg of the MRGO had little influence on the water levels in the IHNC during Katrina. Our work indicates that it was the confluence of the MR-GO, the ICWW, and the IHNC that resulted in the magnifying effect on the surge heights in IHNC previously identified as the "Funnel Effect". Our work contradicts the IPET finding that the "MRGO was far from the 'hurricane highway' moniker with which it has been branded." Our work indicates that the most important effect of the MRGO was destruction of the wetlands and freshwater swamp growth that had provided natural defenses for this area. This destruction was due directly to the increased salinity introduced by the MRGO and by the widening of the MRGO channel due to ship wave pressure erosion effects. MRGO had additional effects provided by its geometry relative to other waterways and the required dredging associated with maintenance of this channel. So the observation that the MRGO in itself had a small effect on the rise of water levels is correct; however, the conclusions drawn from this observation are not correct. Our work does not corroborate the statement "Of the 50 major breaches experienced by the hurricane protection system during Katrina, ALL (capital letters added for emphasis) but four were due to overtopping and erosion."

Our work indicates that there were many instances in which it is very likely that breaching occurred BEFORE overtopping. This discrimination is important for several reasons; one of the most important reasons regards learning from the failures so that proper corrective measures are introduced into design of repairs and new components. Perhaps the nuance in this IPET statement regards "no evidence of systemic breaching caused by erosion on face or water sides of the levees"; the breaching before overtopping is due to much more than erosion; additional important phemenona such as liquefaction and seepage pressures

must be recognized – in addition to the water velocity magnification effects caused by 'hard structures' adjacent to 'soft' earth structures.

The fact that components in the NOFDS were overtopped and this overtopping exceeded the design criteria (SPH) should not lead to the conclusion that failure or poor performance should be expected. Quite to the contrary; good engineering design would incorporate additional defensive measures including armoring, appropriate factors of safety leading to 'excess' capacity, configuration to provide for acceptable performance (e.g. creation of polders or separated defended sections), ductility (ability to maintain capacity even though damaged, defective, or pushed beyond the capacity), and appropriate correlation (high positive for series components achieved through appropriate uniformity in design, construction, operation, and maintenance; low positive for parallel components to achieve independence in performance characteristics).

Yes, the four breaches in the outfall canals and IHNC involving I-walls occurred before water levels reached the top of the floodwalls, and in most cases well before the water levels reached the design conditions. All of the conditions responsible for these breaches were known and foreseeable in advance of these failures. In this sense, these were 'predictable failures.' For a wide variety of apparent reasons, the knowledge was not used
appropriately.

Yes, transitions in the NOFDS created a large number of vulnerabilities and reduced the effectiveness of the protection. Yes, some of the transitions are associated with changes in the organization, incompletion of the authorized construction, and others associated with necessary penetrations through the system. None of these factors in themselves provides acceptable explanations for the ineffectiveness of the transitions in a system that
was constructed over a period of more than 40 years. It is clear that this system while possibly originally conceived as a coherent and integrated system (1965) the NOFDS was not designed, constructed, nor maintained as such. This system was riddled with major and systemic flaws whose source lies primarily in organizational and institutional dysfunctions. The so-called engineering failures are not the causes of these failures; the causes are firmly rooted in the organizational and institutional dysfunctions.

Yes, some sections of the NOFDS did perform well even though "subjected to design-exceeding conditions and forces." However, this performance was due to many more factors than erosion resistant materials and more conservative

designs. In many cases, natural defenses (wetlands, barrier islands, swamp vegetation) and constructed works (roadways, railways, bridges) also provided important defenses. This is an important observation as it regards development of strategies for future development of the NOFDS as well as for a comprehensive understanding of the reasons for successes and failures in the NOFDS.

Yes, "Flooding from Katrina covered approximately 80% of the New Orleans metropolitan area." While the IPET studies indicate "Approximately two-thirds of that flooding can be attributed to water flowing through breaches" our work indicates figures closer to 75% to 80% and in some cases 90%. This difference seems to be founded primarily in the understanding of breaching after or before the peak of the surge conditions in various parts of the NOFDS. Our work indicates that IF everything that we had in the NOFDS had worked properly and as we wanted it to work, then we would have had some minor flooding; some soaked carpets, missing shingles and window glass (and other associated wind damage). We would not have had the single most catastrophic
failure of an engineered system in the history of the United States. This catastrophic failure developed because of unanticipated, unintentional, and undesirable breaches in the NOFDS. The catastrophe was compounded by rescue and recovery meltdowns.

Yes, pump stations did play a role in the flooding. There were systemic and pervasive flaws in this part of the NOFDS that persist to this point in time. As the future NOFDS is developed, it is essential that the ability to pump water out of the sections or polders that comprise the NOFDS be re-engineered (start over), designed, constructed, maintained, and continually proofed and improved.

Consequences

Yes, the most serious direct impact of the failures of the NOFDS that accompanied Katrina was the high number of deaths (currently estimated at more than 1,700 with more than 500 still missing). Note that the 'blame' for these deaths should not be placed in this statement on 'Katrina'; Katrina severely tested the NOFDS. The NOFDS failed the test. This catastrophe did not result from an act of 'God'. It resulted from acts of 'People.' This represents human failure to deliver appropriate technology to prevent such a catastrophe. The NOFDS failed the test because of a large number of flaws and defects that had been embedded in the system primarily during the period 1965 - 2005; these have been termed engineering failures. However, the engineering failures are not the causes. The

causes are firmly founded in organizational and institutional failures that are primarily focused in the Corps of Engineers.

The background that has been developed during the ILIT study indicates that direct and indirect economic costs of the failure of the NOFDS will approach $400 billions. This cost is about 100 times the cost originally estimated in the Corps of Engineers cost-benefit studies that justified the levels of protection incorporated into the NOFDS. The cost-benefit process that continues to be used by the Corps of Engineers to justify protection expenditures and recommendations to Congress is very seriously flawed. Until these flaws are addressed, decision makers will not have appropriate information to make informed decisions. And, the protected public will not have appropriate information to make decisions on how they want to live their lives in this area.

The ILIT study findings do not provide corroboration of the statement "Overall, any sustained environmental loss from flooding and flood management is indicated to be very small in the context of long-term impacts from development in the region." Many sources have documented that there have been and continue to be substantial environmental losses from flood management as conceived, designed, constructed, and in some cases, operated and maintained (e.g. navigable waterways) by the Corps of Engineers. The legendary environmental insensitivity of the Corps of Engineers is encapsulated in this statement.

Lessons Learned

Resilience

Yes, resilience is clearly needed in all future flood defense systems, and particularly for those that protect large metropolitan areas or resources important to such areas. Resilience has been and continues to be a very important part of engineering many types of important systems. There is a substantial body of knowledge about how resilience, robustness, and fail-safe design can be achieved. This body of knowledge needs to be properly understood by the Corps of Engineers and this knowledge appropriately incorporated into revisions of ALL guidelines involving engineering, construction, operation, maintenance, and continued improvement of future flood defense systems. The Corps of Engineers must have highly qualified and motivated engineers and assisting contractors that can help translate these guidelines to the realities required to achieve and maintain a high quality and reliable NOFDS.

Systems Performance

Yes, detailed attention needs to be given to life-cycle performance of the NOFDS. It must be understood that 'systems' consist of many highly interactive, inter-dependent, and adaptive components that include the 'operators', organizations and institutions, hardware and structures, procedures and processes, environments, and interfaces among the foregoing. Systems design and engineering is another technology that needs to be properly understood by the Corps of Engineers and this knowledge appropriately incorporated into revisions of ALL guidelines involving engineering, construction, operation, maintenance, and continued improvement of future flood defense systems. The Corps of Engineers must have highly qualified and motivated engineers and assisting contractors that can help translate these guidelines to the realities required to achieve and maintain a high quality and reliable NOFDS.

One of the most important parts of the 'system' is the Technology Delivery System (TDS) upon which the physical components in a future NOFDS are founded. This TDS represents the collation of the public, government, and industry / commerce whose goal is provision of adequate and acceptable long term hurricane and other (river, rainfall) flood protection - the NOFDS. These components are embedded in and interact with their natural and cultural environments. Inputs comprise knowledge plus human, natural, and fiscal resources. Outputs include desired goods or services and undesired outcomes or unintended consequences. While the physical components in the NOFDS clearly need rehabilitation and dramatic improvement, this goal can not be realized unless the TDS is also rehabilitated and improved. Perhaps the most important cause for failure of the NOFDS during and after hurricane Katrina was failure of the TDS that was responsible for its lifecycle (concept development, design, construction, operation, maintenance, improvement). The Corps of Engineers had a primary role in this dysfunctional TDS; but, there were other organizations that contributed to this dysfunctional performance. The TDS must be re-engineered and actually translated to actions and behaviors and coupled with appropriate and sustained resources before a desirable and acceptable NOFDS can be realized.

Risk and Reliability

Yes, modern risk and reliability approaches hold much promise for providing information and insights that can help a parts of the TDS make better decisions. It is important that holistic and realistic proactive, reactive, and interactive approaches be utilized and measures taken to not allow these approaches to

become paper chases or numbers games. In implementation of these approaches it must be remembered that the greatest sources of
uncertainties are associated with human factors (operator and organizational performance, knowledge acquisition and utilization). Experience with a wide variety of high risk and reliability systems clearly indicates that creation and maintenance of High Reliability Organizations (HROs) is the single most important approach that is needed to develop engineered systems that have desirable and acceptable quality (serviceability, safety, compatibility, durability) over their lifetime. The steps taken and that are being taken by NASA to translate itself to an HRO (the 'One NASA Initiative') are one model that should be studied as one considers a future TDS for the NOFDS.

Knowledge, Technology, and Expertise

Yes, "The history of the planning, design, and performance of the Hurricane protection System in New Orleans points out a dilemma."

Yes, the pieces were not put together in a meaningful. Although the 'dots' were clear, they were not connected. And, there are many examples that developed throughout the history of the NOFDS between the disastrous flooding of the system during hurricane Betsy (1965) and the catastrophic flooding of the system during hurricane Katrina (2005). These examples are clearly indicative of a 'non-learning' organization. The ILIT study indicates that there are many reasons for this non-learning characteristic that includes loss of core competencies in the Corps of Engineers ("we have taken engineering out of the Corps of Engineers"). Bureaucratic engineering produced a defective and deficient NOFDS. In many cases, we found evidence of 'lethal arrogance'; information that provided potential early warnings was rejected because "the Corps knows everything." But, there were other important players that include the executive and legislative branches of both the federal, state and local government agencies. The ILIT study choose to focus these breakdowns and malfunctions in the 'dysfunctional TDS'. It is clear that the TDS must be rehabilitated before an adequate long-term NOFDS can be realized.

ILIT Findings: Failure of the NOFDS

Failure of the NOFDS was a predictable, predicted, and preventable catastrophe. Failure of the NOFDS was not caused by an overwhelming extreme natural event. While portions of the NOFDS were overtopped by hurricane Katrina's surge and waves, the majority of the flooding came from unanticipated and unintended breaches in the levees and the floodwalls and water entering through and over low spots in the NOFDS system. The roots of these unanticipated and

unintended developments are firmly embedded in malfunctions in the TDS. Eight categories of TDS malfunctions were identified by the ILIT that played primary roles in the failure of the NOFDS. These malfunctions were corroborated with results from other in-depth studies performed and documented by federal agencies (White House, Congress), public groups, and individual investigators.

Failures of foresight:
Catastrophic flooding of the greater New Orleans area due to surge from an intense hurricane was predicted for several decades. The consequences observed in the wake of hurricane Katrina were also predicted. The hazards associated with the NOFDS were not recognized, defensive measures identified and prioritized, and effective action was not mobilized to effectively deal with the hazards.

Failures of organization:
The roots of the failure of the NOFDS are firmly embedded in flawed organizational - institutional systems. The organizational - institutional systems lacked centralized and focused responsibility and authority for providing adequate flood protection. There were dramatic and pervasive failures in management represented in ineffective and inefficient planning, organizing, leading, and controlling to achieve desirable quality and reliability in the NOFDS. There were extensive and persistent failures to demonstrate initiative, imagination, leadership, cooperation, and management.

Failures of resource allocation:
Contributing to the failure of the NOFDS was provision of inadequate resources based primarily on recommendations provided by the Corps of Engineers. This was followed by
failure of the federal and state governments to fund badly needed improvements once limitations were recognized. In several instances, State agencies pressured for 'lower cost' solutions not realizing that these solutions would result in lowering the quality and reliability of the NOFDS. There were important deficiencies in the cost - benefit analyses used to justify the levels of protection and their continued improvement as knowledge and technology advanced.

Failures of diligence:
Forty years after the devastating flooding caused by hurricane Betsy, the flood protection
system authorized in 1965 and founded on the Standard Project Hurricane (SPH) was not completed. The concept and application of the SPH was recognized to be

seriously flawed, yet there were no adjustments made to the system before Katrina struck. Early warning signs of deficiencies and flaws persisted throughout development of the different components that comprised the NOFDS and these signs were not adequately evaluated and acted upon.

Failures of decision making:

The history of this system was marked by a series of flawed decisions and tradeoffs that proved to be fatal to the ability of the system to perform adequately. Compromises in the ability of this system to perform adequately started with the decisions regarding the fundamental design criteria for the development of the system, then were propagated through time as alternatives for the system were evaluated and engineered. Design, construction, operation, and maintenance of the system in a piecemeal fashion allowed the introduction of additional flaws and defects. Efficiency was traded for effectiveness. Superiority in provision of an adequate NOFDS was traded for mediocrity and getting along.

Failures of management:

Requirements imposed on the Corps of Engineers by Congress, the White House, State and local agencies, and the general public have changed dramatically during the past three decades. Defense, re-construction, maintenance, waste disposal, recreational, emergency response, ecological restoration have served to divert attention from flood control. Public and Congressional pressures to reduce backlogs of approved projects, improve project and organizational efficiency (down-sizing, out-sourcing), address environmental impacts and develop appropriations for projects have served to divert attention from engineering quality and reliability of flood control. Engineering technology leadership, competency, expertise, research, and development capabilities appear to have been sacrificed for improvements in project planning and controlling.

Failures of synthesis:

While individual parts of a complex system can be adequate, when these parts are joined

together to form an interactive - interdependent - adaptive system, unforseen failure modes can be expected to develop. These unforseen, but forseeable, failure modes did develop in the NOFDS during hurricane Katrina. It is evident that insufficient attention was given to creation of an integrated series of components to provide a reliable NOFDS. Synthesis was subverted to decomposition. As a result, many failures developed at interfaces or 'joints' in the NOFDS.

Failures of risk assessment and management:
The risks (likelihoods and consequences) associated with hurricane surge and wave induced flooding were seriously underestimated. There was inadequate recognition of the primary contributors to the likelihoods and consequences of catastrophic flooding. Sufficient defensive measures to counteract and mitigate these uncertainties were not used. Factors of safety used in design of the primary elements in the NOFDS were not sufficient. Quality assurance and control measures invoked during the life of the system failed to disclose critical flaws in the system. Inappropriate use was made of existing engineering technology available to design, construct, operate, and maintain a NOFDS that would have acceptable quality and reliability. Deficient risk management methods were used to allocate resources and impel action to properly manage risks. Risk management failed to employ continuing improvement, monitoring, assessment, and modifications in means and methods which were discovered to be ineffective.

ILIT Findings: Future NOFDS
Engineering Developments
The technology exists that can be used to develop a NOFDS that will be effective and efficient. A major challenge is timely and proper application of this technology.

Recommendation 1: Develop an integrated and coherent Flood Defense System for the greater New Orleans area (NOFDS) that will provide desirable and acceptable levels of flood protection throughout its life-cycle. Particular attention must be paid to interfaces and interdependencies in this system. The NOFDS should be balanced, complete, cohesive, clear, consistent, and have controls and continuity. The NOFDS should be based on the best available and safest technology and most up-to-date legal standards. Risks should be properly identified, contained and compartmentalized. The system must recognize the unique natural environmental setting including its geology, meteorology, oceanography, the Mississippi River floodplains, deltas and wetlands, subsidence, and the rise in sea level and frequency and intensity of hurricanes. The system must also recognize and accommodate the unique societal and cultural environments of this area.

Recommendation 2: Develop a NOFDS based on enhancing natural defenses supplemented with engineered defenses that incorporate concepts of defenses in depth, robustness or resilience, and fail-safe performance. Selective re-establishment of natural coastal defenses and wetlands and restored floodplains

to provide for river floods should be supplemented with engineering works that together will have the capabilities of providing desirable and acceptable levels of flood protection. Coastal management must be focused on providing safety from flooding and environmental protection. Water should be given space. Some areas will have to be returned to nature and judicious and wise decisions reached on which areas will be populated and developed and the levels of protection that will be provided to these areas. Engineering works should include raising and strengthening and defense of levees, provision of floodgates, storm surge barriers, positioning and defense of modern pump stations, compartmentation to limit potential flooding, adequate and effective evacuation measures to help limit effects on people and their possessions. A robust NOFDS will require a combination of appropriate configuration of engineered elements and components, ductility or an ability to deform and stretch and not loose important performance characteristics, excess capacity so that if some elements or components are overloaded or do not perform desirably then desirable protection can be maintained, and appropriate correlation or mutual relationships so that desirable protection is realized. Fail safe characteristics should be provided in all of the important elements of the NOFDS so that when the design and ultimate performance conditions are exceeded, the performance characteristics are not appreciably compromised.

Recommendation 3: Develop a NOFDS founded on advanced Risk Assessment and Management principles for all phases in the life-cycle including concept development, design, construction, operation, and maintenance. These principles should address natural, analytical modeling, human and organizational performance, and knowledge acquisition and utilization uncertainties and be based on proactive, reactive, and interactive risk assessment and management approaches. These approaches should be based on reductions in likelihoods of failure, reduction in the consequences associated with potential failures, and increases in detection and correction of developments that can lead to failures. Advanced Risk Assessment and Management approaches should be used to provide decision makers with information to define what levels of protection should be
provided for which areas to be protected and how much can and should be spent for those purposes.

Recommendation 4: Develop updated engineering guidelines and procedures for all elements and components to be incorporated in the FDS for all life-cycle phases based on proven state-of-practice and state-of-art technology. Where technology gaps are identified, then substantial development programs should

be implemented to fill these gaps with existing research results. Where technology gaps can not be filled with existing research results, then research should be undertaken or sponsored to enable timely filling of the technology gaps.

Recommendation 5: Develop, implement, and enforce advanced Quality Assurance and Quality Controlmethods and procedures for all life-cycle phases of the NOFDS. Quality Assurance (proactive) and Quality Control (interactive) measures are of particular importance to help disclose 'predictable surprises' and variances in the desirable quality characteristics of the elements and components in the NOFDS. These methods and procedures should be used in all life-cycle phases of the NOFDS including concept development, design, construction, operation, maintenance, and continued improvement. These procedures and measures need to assure that the best available and safest technology is being used and used properly.

Technology Delivery System Developments

The primary requirement for reconstitution of a Technology Delivery System that can and will provide an adequate and acceptable NOFDS is mobilization of the 'will' to provide such a system. If the United States decides that the catastrophe of Katrina will not be repeated, then the necessary leadership, organization, management, resources, and public support must be mobilized to assure such an outcome. One of the primary challenges is time, the clock is ticking until this area of the United States is again confronted with a severe challenge of flooding.

Recommendation 1: Seriously consider defining risk in the framework of federal, state, and local government responsibilities to protect their citizens.

Recommendation 2: Exploit the major and unprecedented role that exists for citizens who should be considered part of governance in the spirit that those who govern do so at the informed consent of the governed. This is the population exposed to catastrophic risks and the people that will be protected by the NOFDS. Authorities for catastrophic risk management should ensure that those vulnerable have sufficient and timely information regarding their condition and a reciprocal ability to respond to requests for their informed consent especially regarding tradeoffs of safety for cost. The public protected by the NOFDS need to be encouraged to actively and intelligently interact with its development.

Recommendation 3: Intensify, focus, and fund Corps of Engineers modernization efforts increasing in-house engineering capabilities and project performance, increasing in-house research and development capabilities, increasing in-house engineering performance of technically challenging projects, developing an organizational culture of high reliability founded on existing cultural values of Duty, Honor, Country, and developing a leadership role and responsibility for technical and management oversight of all phases in development of a NOFDS. Technical superiority must be re-established. Outsourcing must be balanced with in-sourcing to encourage development and maintenance of superior technical leadership and capabilities. This will require close and continuous collaboration of federal legislative, executive, and judicial agencies. This will require that
the Corps of Engineers reconceptualize itself as a pivotal part of a modular organization developing partnerships with other federal agencies, state and local governments, enterprise interests, and private stake holders.

Recommendation 4: Restructure federal/state relationships in flood control. One possible model is what has been called "modularity" -- a concept which involves provisional and functional rearrangement of units in terms of alternative configurations of tools, structures and relationships.

Recommendation 5: Develop a National Flood Defense Authority (NFDA) charged with oversight over the design, construction, operation and maintenance of flood control systems. Each state would have an equivalent organization that could foster cooperation and developments between and within the states. The Corps of Engineers, state flood control authorities, and technical advisory boards would work with the NFDA to foster application of the best available technology and help coordinate development and maintenance efforts and planning. In cooperative developments, federal and state governments would provide reliable and sustainable funding for the life-cycle of specific flood defense systems. This development should be accompanied by development of an integrated and coherent Louisiana Flood Defense Authority representing state, regional, local, city, and public stakeholders that can focus and prioritize stakeholder interests and requirements and collaborate with the Corps of Engineers in development of a NOFDS.

Recommendation 6: Because of the importance of emergency response in the NOFDS, FEMA should be developed as a high reliability organization and returned by the executive branch to Cabinet level status. A new Council for Catastrophic Risk Management should be appointed in the White House and

given oversight of disaster preparation and response. A similar body should be appointed to Congress. Incentives must be created to encourage all levels of government to seriously deal with potential national, regional, and local catastrophes.

Summary

This has been the most challenging study of the failure of an engineered system that I have been involved in during my 53 year engineering career. I have been directly involved in investigations of more than 100 high consequence failures and performed detailed studies of more than 500 additional such failures (including work for NASA on the Columbia accident and the shipping industry and USCG on the Exxon Valdez). This failure played out exactly as the majority of these other failures played out. The dominant contributors to the failure were centered in human and organizational malfunctions represented as breakdowns in the TDS. The failure developed during the operations and maintenance phases of the system. The vast majority of the contributors to the failure were firmly rooted in the early concept development and design phases of the system. I had particularly good qualifications to help perform this study. I started my career in 1954 working for the Corps of Engineers in the south Florida flood control district helping engineer and construct levees, pump stations, and canals around Lake Okeechobe. My father was a career Corps of Engineers officer (started at St. Louis District and retired at Jacksonville District). I was raised Corps of Engineers; 'yes sir and salute.' I left the Corps of Engineers in 1959 and returned to school. I joined Shell Oil Company in 1960 and worked in the coastal areas in Mississippi, Alabama, Louisiana and Texas. By 1965 I headed Shell Oil Company's Head Office Offshore Civil Engineering Group in New Orleans (registered civil, structural, geotechnical engineer in 8 states). In September 1965 after hurricane Betsy passed over New Orleans, I returned to our home after we (my wife and 6 month old son) had evacuated at the height of the storm to my central New Orleans office (where we lived for the next 2 months). I swam and waded back to the site of our flooded home. Ten feet of water. We lost everything including our home and wedding photographs. There was no help. There was no insurance. Bank account equals zero time of our lives. But, our lives went on. After having worked in all of the major coastal and offshore areas worldwide (engineering, construction, research, operations, maintenance, management), I left PMB-Bechtel (Vice
President) and industry in 1989 and was appointed as a Professor of Engineering at the University of California Berkeley. Since then, my work has focused on risk assessment and management, engineering systems, and engineering and project management. So, when I returned to New Orleans in September 2005 to help the

ILIT with the field studies of the failures of the NOFDS, I was coming home to something I had seen 40 years before. Only this time there was vastly more destruction, devastation, and grief. And this time, I saw the world through a different set of glasses. I returned to the site of our old home in East New Orleans (Pines Village); a new home had been rebuilt on the foundations of our previous home. The owners were dragging wet mattresses out of their home just as I had done 40 years earlier. I broke down and cried. Mine were tears of frustration because by that time I understood that this was a predictable and preventable catastrophe. Flooding of this area had occurred in almost exactly the same ways and from the same places 40 years earlier. This catastrophe involved much more than the physical failures of levees, floodwalls, and pump stations; it had it roots firmly embedded in human and organizational performance malfunctions (errors) - breakdowns in the TDS that we had relied upon to provide adequate flood protection.

Bob Bea

Appendix 7- Seepage and Lane's Weighted Creep Ratio Calculations

Van Zyl and Harr (1981) classified seepage erosion failures into three modes: heave, piping and internal erosion. In order to investigate the hydraulic features of a weir or dam (levee), especially seepage, extensive field studies were made by Bligh (1910) on weirs in Egypt and India. However, Bligh's method was too conservative and was later modified by Lane (1935) on the basis of a study of about 280 existing dams on pervious foundations, 24 of which had failed. The "line of creep" theory is based on the conclusion that the "line of creep", i.e., the line of contact between the dam (levee) and cutoffs (sheetpile) with the foundation will produce less resistance to percolation than another path through the foundation material or substrate (National Engineering Handbook 1968).

Thus Lane believed that the majority of dam failures he had investigated were due to piping along the line of creep. Lane's method incorporates an assumption that the resistance to flow through the soil, and therefore the rate of pressure drop, will be three times higher in the vertical than in the horizontal direction. One explanation put forward is that horizontal layering in the soil provides an easier passage to seepage. Thus in Lane's method, vertical structural surfaces are weighted so that they become three times as effective in reducing seepage pressure as horizontal ones. Many researchers have pointed out that this assumption only hold true in a limited set of soil conditions. It certainly does not hold true in very sandy substrate, especially well rounded and sorted beach sands, such as typifies the Pine Island sands beneath the London Avenue Canals in New Orleans.

For this reason Lane's should be used with caution and this and other empirical methods are commonly used in the routine design of low- to medium head structures. Lane's Theory is simple and rapid to apply so it is convenient for making quick preliminary analyses of a structure's overall resistance to seepage. However, the method does not provide any check against the danger of excessive pressure gradients developing locally at the downstream end of the structure. It also appears to imply that a vertical cut-off would be equally effective whether located at the upstream or downstream end of a structure. More detailed analytical methods show this is not the case. It should be remembered that the line-of-creep approach is empirical and like every empirical approach leads to a design with an unknown factor of safety.

Lane's creep ratio is determined by a so-called safe seepage Length L divided by h the head of water, i.e., L/h. L is defined as the sum of lengths of vertical flow paths plus one third of the sum of horizontal flow paths with surface sloping less than 45 degrees being considered horizontal; sloping greater than 45 degrees being vertical, so

L = sum of lengths vertical flow paths plus 1/3(sum of horizontal flow paths).

Lane (1935) then gives a table of recommended or safe creep-head ratios L/h. So for very fine sand or silt if the ratio is greater than 8.5 the design is considered safe; for fine sand the ratio needs to be greater than 7.0; for medium sand greater than 6.0; and for soft clay 3.0 (Adapted from USBR 1974, Untitled Table, Page 364).

Eustis Engineering (1986 and 1993) used the Lane's Creep Ratio in some of their calculations for London Ave canal. First they determined the Lane's creep ratio for the various reaches and then used the creep ratio as a check on the sheet pile lengths (Figures 7.12 and 7.14). At the I-wall reach at the Mirabeau breach Eustis state the head is 17.9 feet, which appears to be the 13.9 height of the wall added to the fact that the land behind the levee on the protected side is at minus 4 feet, giving h = 17.9 feet. They used a weighted creep length of 57 feet and determined a Lane's creep ratio of 3.2. So how long must the sheet pile have been to come up with a ratio of 3.2? As pointed out above the quantity L = sum of the vertical lengths plus 1/3 of the horizontal. The width of the levee at the fill marsh surface is 45 feet (Figure 7.13). To obtain a ratio of 3.2 there would have to have been a vertical flow path of 39 feet (57-55/3 = 57-18 = 39). Thus they must have done this calculation assuming a sheet pile length of 18.5 feet below the base, which is at minus 4; thus the sheet pile must have extended down to -22.5 feet NGVD29. If there was no sheet pile then the levee would have had to be 57*3 = 171 feet wide

Assuming the material to be soft clay then the creep ratio they calculated exceeds Lane's recommendation by 6%, which would be cutting it very, very close. However they fail to recognize that they are only taking into account the thin marsh/clay layer above the very thick sands (Figure 7.13). As any geologist would know, the sands would be the preferred pathway for any seepage. If they had done the same calculations for the sands with a ratio of 8.0 then the weighted creep distance flow path would have been 125 feet too short, thus sheet pile to -66.5 feet NGVD29 would have been required. This is a huge and very significant difference. The sand calculation would have required that the sheet pile be 44 deeper forming an effective seepage barrier.

In a 1993 report (Figure 7.12) Eustis use Lane's Weighted Creep ratio to determine what the sheet pile length should be. For the same Mirabeau reach as calculated above they state they need sheet pile to -6.5 feet. So something has changed between 1986 and 1993! Using sheet pile to -6.5 and the same levee cross section as above (Figure 7.13) gives a Lane's weighted creep ratio of 1.2, which is well below that recommended by Lane for soft clays, let alone for sands.

As pointed out above Lane's method has been widely used although there are obvious problems with some of its assumptions. First, flow configuration and gradients are not affected by the soil type (Kashef 1986) in homogeneous and isotropic media under steady–state flow. The gradients are affected by the quantity h, and so are the uplift pressures. The National Engineering Handbook (1968) categorically states that so many intermediate variables affect the design for safety against seepage or piping that a large factor of safety is mandatory. When one considers that the hydraulic conductivity can vary from 10 cm/sec for coarse gravel to 10^{-9} cm/sec for dense clay, the complexity of the problem is apparent. If different materials are present in the subsurface or foundation, the material having the largest creep ratio should be considered as the foundation material and the design made accordingly. It is almost presumptuous to point out (Nation Engineering Handbook 1968) that adequate foundation investigations are mandatory for safe design of structures that hold back water such as levees and flood walls.

Uplift or Heave Forces

Soil type, rate of head increase and the flow condition are the main dependents for modes of seepage erosion failure (Van Zyl and Harr, 1981). The soil type controls whether heave is followed by a quick condition as in clean sand or whether heave leads to crack formation, concentrated flow and piping. Heave, leading to cracks, concentrated flow and piping, appears to be more common in granular soils with a large percentage of fines. A rapid increase in head may result in heave of the surface, leading to a quick condition (Van Zyl and Harr, 1981). This could be a typical failure condition on the downstream side of a levee bounded canal being filled rapidly, such as occurred during Katrina.

Heave is analyzed by comparison of seepage force per unit volume with effective unit weight of selected critical volume of soil. Terzaghi (1929) presented an exit gradient approach to seepage analysis in his classical work on failure of dams by seepage erosion. His theoretical development was based on the summation of the vertical seepage forces exerted by the upward flow of water and the vertical

downward weight of the submerged soil. However there is an even simple method.

It is imperative in the design of levees and flood walls to determine the uplift forces on the downstream (protected side) to ensure that these do not exceed the load (weight) capacity of the floor and thus impact the overall stability. The ratio of the downward forces to the uplift forces should be greater than 1.0 by an appropriate margin of safety. Kashef (1986) amongst others points out that the factor of safety in hydraulic structures should not generally be less than 4. So just what was the ratio for the London Avenue canal flood wall at Mirabeau prior to the breach?

A very simple calculation allows one to determine the ratio between uplift vs. downward forces on the protected side of the levee. Recognizing that the thick well sorted beach sands beneath the floodwall would rapidly allow seepage (as shown by the Lane's creep calculations for example), the pressure at the base of the clay layer/top of the sand in the canal due to the surge can be calculated as the head * the density of water = hsurge+depth to interface x 62.4 lbs/cub ft = 1280 psf. On the protected side 6 feet of marsh and clay overlies the sand layer, so the gravity(weight) downward force = 6 feet * density of the soil which is given as 110 ft/cubft = 660 psf. So the upward pressure far exceeds the downward, the ratio being 0.5. As pointed out this should be at least 4 so it is easy to see that the system as built would fail, there being no cutoff (sheetpile) as a hydrologic/seepage barrier in the sands.

The Corps of Engineers has done extensive studies on seepage, piping and sand boils (APPENDIX 8). Additionally, the levee designers could easily have found information on seepage and creep ratios in PCA (1980), and USACE EM 1110-2-1901 (1993). The applicability of Lane's Creep ratio for a specific structure should always be questioned and only used as a rule of thumb. Given that thousands of lives depended on the levee system, additional seepage and piping analyses using more rigorous methods (e.g. flow nets) should have been undertaken to assess hydraulic gradients, uplift pressures, and seepage flows. Measures to control seepage normally include provisions to increase the length of the seepage path and otherwise reduce seepage flows, and provisions to filter and drain the seepage water. Given the high importance of controlling seepage on the safety and stability of the levees, adequate redundancy should be incorporated when designing seepage control measures.

The above review using simple techniques available to the engineers who designed the London Ave canal levees proves that the designs were inappropriate and the factor of safety based on uplift forces was 0.5! No wonder the system failed. Miscalculations like this are inexcusable.

Development of Underseepage and Sand Boils

We believe that the failure at Mirabeau resulted from uplift forces cracking the overlying clay, the subsequent development of large sand boils which expanded rapidly into a large scour hole into which the levee flood wall collapsed. This failure mechanism has been described before for the Mississippi R iver system. Turnbull and Mansur (1961) explained underseepage mechanisms and sand boil formation at Mississippi River levees as a result of the studies and investigations conducted by the Army Corps of Engineers covering a period of 1937 to 1952:

> *"Whenever a levee is subjected to a differential hydrostatic head of water as a result of river stages higher than the surrounding land, seepage enters the pervious substratum through the bed of the river and riverside borrow pits or the riverside top stratum or both, and creates an artesian head and hydraulic gradient in the sand stratum under the levee. This gradient causes a flow of seepage beneath the levee and the development of excess pressures landward thereof. If the hydrostatic pressure in the pervious substratum landward of the levee becomes greater than the submerged weight of the top stratum, the excess pressure will cause heaving of the top blanket, or will cause it to rupture at one or more weak spots with a resulting concentration of seepage flow in the form of sand boils."*
>
> *"In nature, seepage usually concentrates along the landside toe of the levee, at thin or weak spots in the top stratum, and adjacent to clay-filled swales or channels. Where seepage is concentrated to the extent that turbulent flow is created, the flow will cause erosion in the top stratum and development of a channel down into the underlying silts and fine sands, which frequently exist immediately beneath the top stratum. As the channel increases in size or length, or both, a progressively greater concentration of seepage flows into it with a consequent greater tendency for erosion to progress beneath the levee."*
>
> *"The amount of seepage and uplift hydrostatic pressure that may develop landward of a levee is related to the river stage, location of*

seepage entrance, thickness and perviousness of the substratum and of the landside top stratum, underground storage, and geological features. Other factors contributing to the activity of the sand boils caused by seepage and hydrostatic pressure are the degree of seepage concentration and the velocity of flow emerging from the boils."

References

Lane, E.W. (1935)."Security from under-seepage-masonry dams on earth foundations," *Transactions American Society of Civil Engineering,* 100, pp. 1235-1351.

Publication Number: EM 1110-2-1901
Title: Engineering and Design - Seepage Analysis and Control for Dams with CH 1
Proponent: CECW-EG
Publication Date: 30 September 1986 (original) 30 April 1993 (change 1)

U.S. Bureau of Reclamation, 1974, Design of small dams: U.S. Department of the Interior, Bureau of Reclamation, Water Resources Technical Publication, 816 p.

APPENDIX 8- Previous Studies on Levee Underseepage Conducted by USACE

The first investigation of potential levee underseepage was initiated by the USACE Mississippi River Commission in 1937 in response to problems caused by high water conditions. More detailed study was carried out by the USACE Waterways Experiment Station (WES), Vicksburg, MS in the 1940's. Procedures to evaluate the quantity of underseepage, uplift pressures and hydraulic gradients were developed based on closed-form solutions for differential equations of seepage flow presented by Bennett (1946). In 1956, a technical memorandum, TM 3-424 was published by the USACE Waterways Experiment Station documenting the analysis of underseepage and design of control measures for Lower Mississippi Valley levees (Mansur *et al.* 1956). In this document, the top stratum landside of levees is classified into one of three categories:

(1) no top stratum;
(2) top stratum of insufficient thickness to resist hydrostatic pressures that can develop; and
(3) top stratum of sufficient thickness to resist hydrostatic pressures that can develop during the maximum design flood.

Kolb (1976) discussed underseepage data collected by USACE Vicksburg District along a randomly selected 40-mile reach of the river during the 1973 flood. He pointed out the most dangerous top stratum category as the second category listed by Mansur *et al*., 1956. In this category, artesian pressures can build up beneath the top stratum landside of the levee to a range of 25% to 75% of the net head on the levee, and may extend significant distances landward of a levee.

Mansur *et al.* (1956) classified seepage as heavy, medium and light. Turnbull and Mansur (1961) presented seepage conditions and upward gradients through the top stratum measured by piezometers during the 1950 high water (Table 2.1). During the high water of 1950, sand boils were observed in a hydraulic gradient range of 0.5 to 0.8. In developing these seepage conditions, sites were eliminated where the top stratum thickness was less than 5 ft or greater than 15 ft (Technical Letter, ETL 1110-2-555, 1997).

Turnbull and Mansur (1961) summarized the design and analysis procedure of levees presented in TM 3-424. Department of Army published in 1978 (and updated in 2000) an Engineer Manual (EM) 1110-2-1913 "Design and Construction of Levees".

Other than advanced numerical modeling, this Engineer Manual represents the state-of-practice analysis method for evaluating hydraulic gradient due to levee underseepage (Gabr *et al*., 1996).

The Army Corps of Engineers investigated possible remedial measures to underseepage problems, which are discussed below. The most common underseepage control measures include pressure relief wells, landside seepage berms, riverside blankets, drainage blankets or trenches, cutoffs, and sublevees. Muskat (1937) presented a design methodology for relief wells. Middlebrooks and Jervis (1947) revised Muskat's method to include partial penetration of the relief wells. Barron (1948) presented a design methodology for fully penetrating relief wells. The Department of the Army published Engineer Manual (EM) 1110-2-1905 "Design of Finite Relief Well Systems" in 1963 and EM 1110-2-1914 "Design, Construction, and Maintenance of Relief Wells" in 1992. Mansur *et al*. (1956) stated that pressure relief wells, riverside blankets, and landside seepage berms are generally applicable for Mississippi River levees. Sublevees and drainage blankets or trenches are applicable in certain special situations.

Wolff (1974) and the U.S. Army Engineer District, St. Louis (1976) studied the performance of 200+ mile levee system along the middle Mississippi River from Alton to Gale (Fig. 2.7). It was reported that the use of the Corps method outlined in Engineer Manual (EM) 1110-2-1913 resulted in a reliable design of levees. It was also concluded that the existing procedure has deficiencies in characterization of a two layer subsurface profile and the inability to model levee bends at corners. Cunny (1980) summarized piezometer data for levees in the Rock Island District, Illinois. Cunny reported that the probability of sand boil occurrence increases with geologic discontinuities. Daniel (1985) reviewed Cunny's report and the other Rock Island data and found that sand boils were bserved at gradients ranging from 0.54 to 1.02. A similar statement was also reported earlier in TM 3-424.

Wolff (1987) studied the application of numerical methods to levee underseepage analysis and pointed out the advantages of special purpose computer programs over traditional underseepage analysis and general-purpose numerical analysis programs. Wolff (1989) developed the computer program LEVEEMSU for analysis of levee underseepage. LEVEEMSU was also used to analyze actual data at a number of levee reaches and back-calculate field permeability values. Cunny *et al.* (1989) also developed a computer program, LEVSEEP, to perform regular underseepage analysis outlined in EM 1110-2-1913, TM 3-424, EM 1110-2-1602, as well as to calculate reduced seepage quantities with the choice of control

measures including seepage berms, riverside blankets, cutoffs and relief wells. Later, Wolff and Taylor (1991) extended LEVEEMSU to analyze three-layer irregular foundation cases.

Underseepage Analysis Methods

In general, approximate methods of solution to confined flow problems include sketching flow nets, electrical analogs, method of fragments (Harr, 1962), viscous flow models such as Hele-Shaw models, relaxation methods (numerical analysis), and small-scale laboratory models. Advanced numerical modeling and 2-D finite element analysis programs provide sophisticated analysis of seepage flow. Boundary fitted coordinate methods also show promise as a method of analyzing seepage flow problems (Thompson *et al.* 1977; Thompson and Warsi, 1982; Thompson *et al.* 1985, and Hartono, 2002).

The transient effects in seepage have been studied under conditions of partial saturation (EM 1110-2-1901, Seepage Analysis and Control for Dams). The flow in partially or unsaturated soils is considered in a transient state. Therefore, transient effects in seepage are normally studied as the migration of a wetting front into unsaturated soils and variations in hydraulic conductivity according to soil water retention curves. Viscous flow models have been used to study transient flow (EM 1110-2-1901). A viscous flow model was constructed at USACE Waterways Experiment Station (WES) to simulate seepage conditions induced in streambanks by sudden drawdowns of the river level. The results from the model study were compared with field observations, finite difference, and finite element methods (Desai 1970, Desai 1973).

Two and three-dimensional finite element seepage computer programs for confined and unconfined flow problems were developed at WES. Steady-state and transient problems can be solved with these computer programs (Tracy 1973a, Tracy 1973b). Transient problems can be treated as a series of steady-state problems. The studies lead by USACE formed a basis for further development of commonly used finite element seepage programs. GMS/SEEP2D is a 2D finite element model that can be used to model steady-state confined, partially confined and unconfined flow. Another finite element seepage analysis program, SEEP/W performs transient seepage analysis considering hydraulic conductivity and water content changes as a function of pore water pressure. Complex geometries, non-homogenous, and anisotropic soil features can be modeled by these finite element models.

For seepage analysis under levees, U.S. Army Corps of Engineers, Design Guidance on Levees, EM 1110-2-1913, recommends use of numerical analysis models such as LEVSEEP and LEVEEMSU or finite element methods such as CSEEP which include two-layer or three-layer subsurface characterization (ETL 1110-2-555, 1997). The computer program LEVSEEP is based on the modeling of the steady-state flow domain with Bennett's (1946) analytical solutions for underseepage and the method of fragments for cutoff analyses. LEVSEEP provides similar analysis with the hand methods of analysis outlined in EM 1110-2-1913, EM 1110-2-1602, and TM 3-424 (Brizendine *etal*. 1995). LEVEEMSU is based on one-dimensional simplification of the steady-state flow domain using the finite difference method. LEVEEMSU solves Bennett's (1946) differential equation for irregular foundation geometry and non-uniform soil properties.

References

Lane, E.W.(1935)."Security from under-seepage-masonry dams on earth foundations," *Transactions American Society of Civil Engineering*, 100, pp. 1235-1351.

Publication Number: EM 1110-2-1901
Title: Engineering and Design - Seepage Analysis and Control for Dams with CH 1 Proponent: CECW-EG
Publication Date: 30 September 1986 (original) 30 April 1993 (change 1)

U.S.B.R., 1974, Earth Manual: U.S. Dept. of Inter. Bureau Reclamation, A Water Resources Tech. Publ., 810 p.
(U.S. Bureau of Reclamation), 1974, Design of small dams: U.S. Department of the Interior, Bureau of Reclamation, Water Resources Technical Publication, 816 p.

Appendix 9.1

November 22, 2005

Louisiana State University
Center for the Study of Public Health Impacts of Hurricanes
Suite 3221 CEBA Building
Baton Rouge, Louisiana 70803

Attn: Ivor van Heerden, Ph.D.
Director

Re: Result of Non-Destructive Testing and Subsurface Investigation
Levee System
New Orleans, Louisiana
SESI Project No.: B05-157

Dear Mr. van Heerden:

Southern Earth Sciences, Inc. (SESI) has completed work on the referenced project. The results of our non-destructive testing (NDT) program, laboratory analysis, and subsurface investigation are presented herein. Authorization to proceed on this project was given to us on November 9, 2005 by Mr. Ed Preau, P.E. of the Louisiana Department of Transportation and Development.

PROJECT INFORMATION

Southern Earth Sciences, Inc. was directed to perform an investigation of the New Orleans levee system at several locations near breached areas. The purpose of our investigation was to evaluate the lengths of the steel sheet piles embedded in the levee sections as well as to assess subsurface conditions. Our field activities during this phase of the project were performed under the direction of Mr. Billy Prochaska, P.E. during the period of November 9-11, 2005.

FIELD TESTING PROCEDURES

Parallel Seismic/Cone Penetration Test (PS/CPT) methods were used to evaluate the embedded lengths of the steel sheet piles on this project. The non-destructive PS/CPT method (patent pending) involves advancing a bi-axial geophone seismic piezocone into the soil adjacent to the member of interest. The member is impacted with an instrumented hammer and the wave energy is measured by the seismic piezocone and recorded on a computer. The resulting plot of wave energy arrival time vs. depth is then analyzed to determine the member length. A complete description of the PS/CPT testing methodology is included in the Appendix.

A Geoprobe 6625 electronic CPT rig was used to perform the non-destructive testing and subsurface investigation. This lightweight rig weighs only about 4.5-tons yet has a full 20-tons pushing capacity. Automated anchors are augered into the soil to provide the necessary reaction for advancing the cone. Photographs of the lightweight CPT rig are included in the Appendix.

Inherit with the PS/CPT test method is the ability to determine subsurface conditions using the seismic piezocone. An instrumented cone attached to a series of rods is continuously pushed into the soil at a constant rate. Built-in load cells are used to continuously measure pore pressure, tip resistance and unit skin friction during penetration where the data is then recorded on a computer.

SUBSURFACE INVESTIGATION

Cone Penetration Tests (CPT)

A total of seven (7) CPT soundings were performed at the levee system to determine subsurface conditions. The CPT soundings were conducted simultaneously with the PS/CPT test process. All soundings were performed in general accordance with ASTM Specification D-5778. GPS coordinates were recorded at all test locations using a hand-held Garmin GPS 72.

The soundings were performed at the approximate locations shown below:

TEST LOCATION	GPS COORDINATES
CPT-01 17th Street Canal Levee	N 30° 00.890' W 90° 07.293'
CPT-02 17th Street Canal Levee	N 30° 00.922' W 90° 07.289'
CPT-03 17th Street Canal Levee	N 30° 00.945' W 90° 07.289'
CPT-04 17th Street Canal Levee	N 30° 00.967' W 90° 07.286'
CPT-05 17th Street Canal Levee	N 30° 00.970' W 90° 07.244'
LCPT-01 London Avenue Levee @ Maribeau	N 30° 00.538' W 90° 04.167'
LCPT-02 London Avenue Levee @ R. E. Lee	N 30° 01.236' W 90° 04.267'

The CPT Log sheets are attached in the Appendix, graphically showing the cone tip resistance, side friction, friction ratio, equivalent N-value and interpreted soil behavior type at each sounding location. The soil behavior type was determined using the methods of Robertson & Campanella (1983).

Also presented in the Appendix are tabulated data sheets for the CPT soundings showing additional soil parameters such as internal friction angle and undrained shear strength. These parameters were derived using a software program developed for SESI and using generally accepted interpretation methods as shown on the tabulated data cover sheets.

In deriving the tabulated soil parameters, a CPT data averaging technique was used over 1-foot intervals. Data Averaging Plots for the CPT soundings are included in the Appendix. These plots shown the actual CPT data recorded in the field (black lines) as well as the 1-foot interval data averaging values (red circles).

The tabulated data is presented for informational purposes only and users should exercise sound judgment when using this information for the interpretation of reliable soil parameters. We highly recommend the engineer be thoroughly familiar with and fully understand the capabilities of the CPT test before using this information for design purposes.

Soil Test Borings

One (1) soil test boring was performed at the B-3 (CPT-03) location to assess subsurface conditions for verification purposes. Samples were obtained at 2-foot intervals to a depth of ten feet using conventional auger drilling methods with our lightweight CPT rig.

Undisturbed samples were recovered from the various cohesive materials with a 3-inch diameter thin walled Shelby tube (ASTM D-1587). The samples were extruded in the field where they were visually classified by the field technician. Representative portions of the samples were then wrapped and sealed to preserve their natural characteristics during transportation to the laboratory for physical testing.

Laboratory tests were performed on the collected samples to determine their physical characteristics. The laboratory test results have been tabulated below:

B-3 (CPT-03) 17TH Street Canal Levee N 30° 00.945' W 90° 07.289'					
DEPTH (FT)	MOISTURE CONTENT (ASTM D-2216)	ATTERBERG LIMITS (ASTM D-4318)			UU TRIAXIAL TEST (ASTM D-2850)
		LL	PL	PI	
2-4	24.9%	73	31	42	1.71 ksf
4-6	20.0%	57	24	33	1.62 ksf
6-8	34.7%	46	26	20	0.51 ksf
8-10	32.9%	54	23	31	0.54 ksf

The laboratory test report sheets for the samples are included in the Appendix.

Upon completion, the boring/sounding holes and anchor hole locations were grouted full depth in accordance with LADOTD regulations.

NON-DESTRUCTIVE TESTING RESULTS

The collected NDT field data was sent to the offices of Olson Engineering, Inc. of Wheat Ridge, Colorado for analysis. Mr. Larry Olson, P.E. is the codeveloper of the PS/CPT process. A copy of the complete PS/CPT analysis results is included in the Appendix.

The PS/CPT results have been summarized below:

TEST LOCATION	GPS COORDINATES	CALCULATED SHEET PILE EMBEDMENT (AVE)
CPT-01 17th Street Canal Levee	N 30° 00.890' W 90° 07.293'	13.9 feet
CPT-03 17th Street Canal Levee	N 30° 00.945' W 90° 07.289'	13.8 feet
CPT-04 17th Street Canal Levee	N 30° 00.967' W 90° 07.286'	13.8 feet
LCPT-01 London Avenue Levee @ Maribeau	N 30° 00.538' W 90° 04.167'	13.6 feet

Embedded lengths are measured from ground surface at each test location.

GENERAL COMMENTS

Professional judgments are presented in this report. These are based partly on our evaluations of field information gathered, partly on our understanding of the characteristics of the project, and partly on our capabilities and experience using CPT and non-destructive testing methods.

Limitations exist for the capabilities of the NDT process used on this project and we do not guarantee the test results in any respect. The NDT results should only be considered an investigative tool for evaluating the conditions of interest and should not be used in place of sound engineering judgment. Accordingly, any overall conclusions and project design decisions should not be based solely on the NDT results presented herein.

We appreciate this opportunity to be of service. If we can answer any questions, please call.

Very truly yours,

SOUTHERN EARTH SCIENCES, INC.

Scott H. Slaughter, P.E.
Branch Manager

cc: Randy Sharma
Billy Prochaska
Ed Preau (2 copies)

SHS/slj

Appendix 1 CPT Log Sheets
Appendix 2 Tabulated Soil Parameters
Appendix 3 CPT Data Averaging Plots
Appendix 4 Laboratory Test Results
Appendix 5 PS/CPT Data Analysis Results (Olson)
Appendix 6 PS/CPT Information Sheet
Appendix 7 Lightweight CPT Rig Photographs

Lab Results

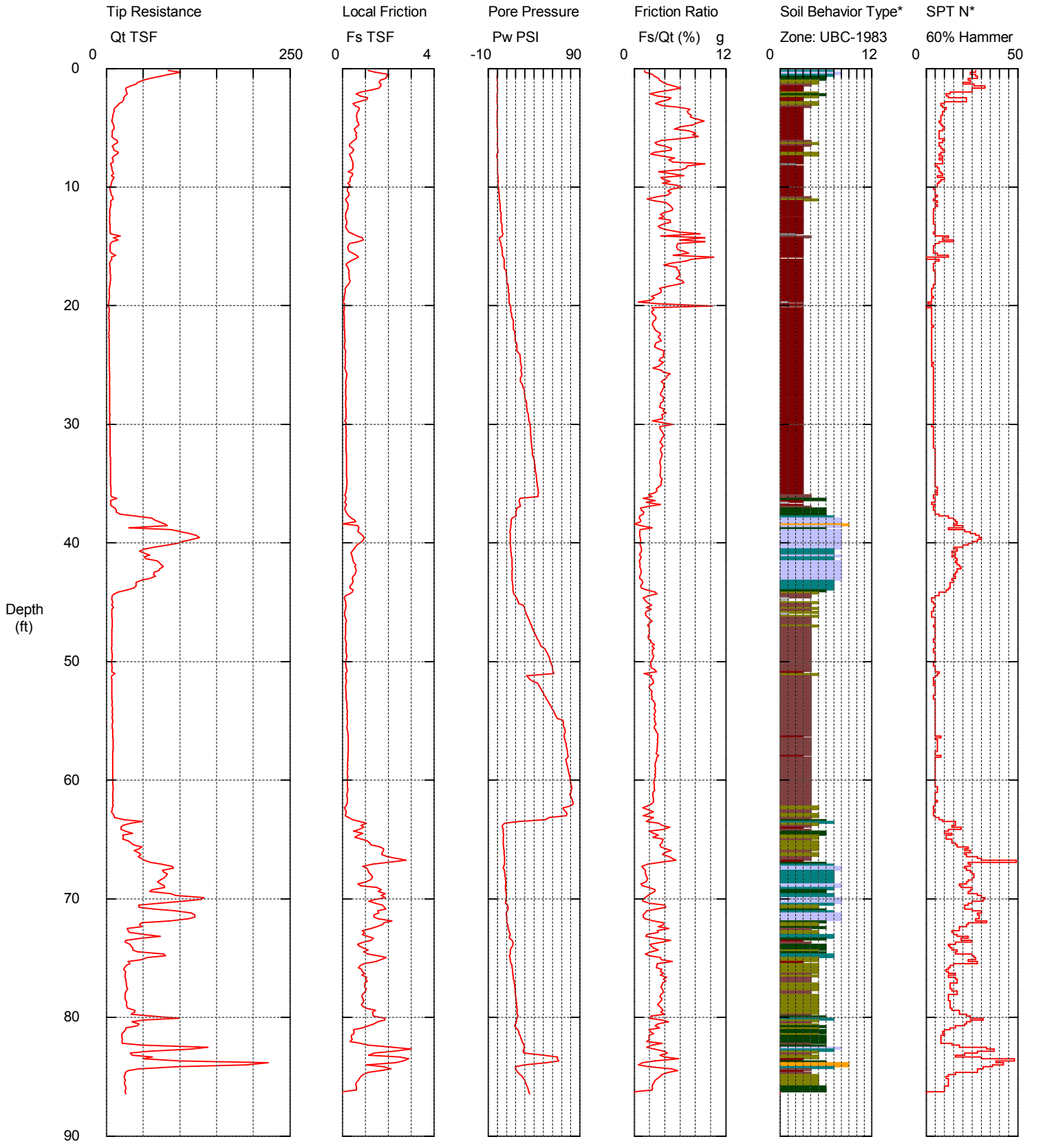
B-3 (CPT-03) 17TH Street Canal Levee N 30° 00.945' W 90° 07.289'					
DEPTH (FT)	MOISTURE CONTENT (ASTM D-2216)	ATTERBERG LIMITS (ASTM D-4318)			UU TRIAXIAL TEST (ASTM D-2850)
		LL	PL	PI	
2-4	24.9%	73	31	42	1.71 ksf
4-6	20.0%	57	24	33	1.62 ksf
6-8	34.7%	46	26	20	0.51 ksf
8-10	32.9%	54	23	31	0.54 ksf

Phase I CPT 01

Southern Earth Sciences, Inc.

Operator: WRP
Sounding: CPT-01
Cone Used: DDG0899

CPT Date/Time: 11/9/2005 2:56:22 PM
Location: 17th Street Canal Levee
Job Number: B05-157



Maximum Depth = 86.45 feet

Depth Increment = 0.164 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

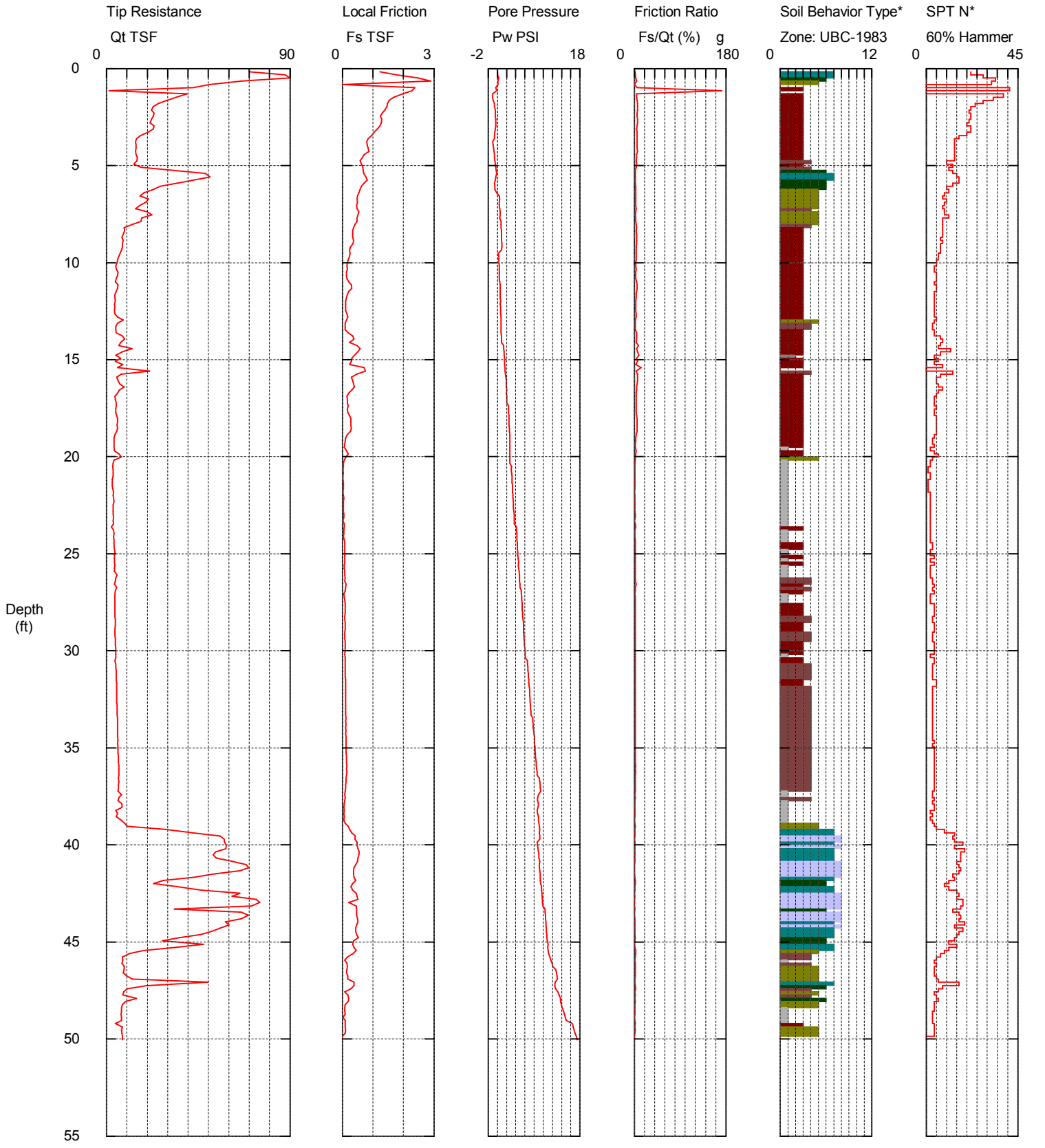
GPS Coordinates
A.IX N30 00.890' W90 07.293'

Phase I CPT 02

Southern Earth Sciences, Inc.

Operator: WRP
Sounding: CPT-02
Cone Used: DDG0899

CPT Date/Time: 11/10/2005 10:44:18 AM
Location: 17th Street Canal Levee
Job Number: B05-157



Maximum Depth = 50.03 feet

Depth Increment = 0.164 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

A.IX

GPS Coordinates
N30 00.922' W90 07.289'

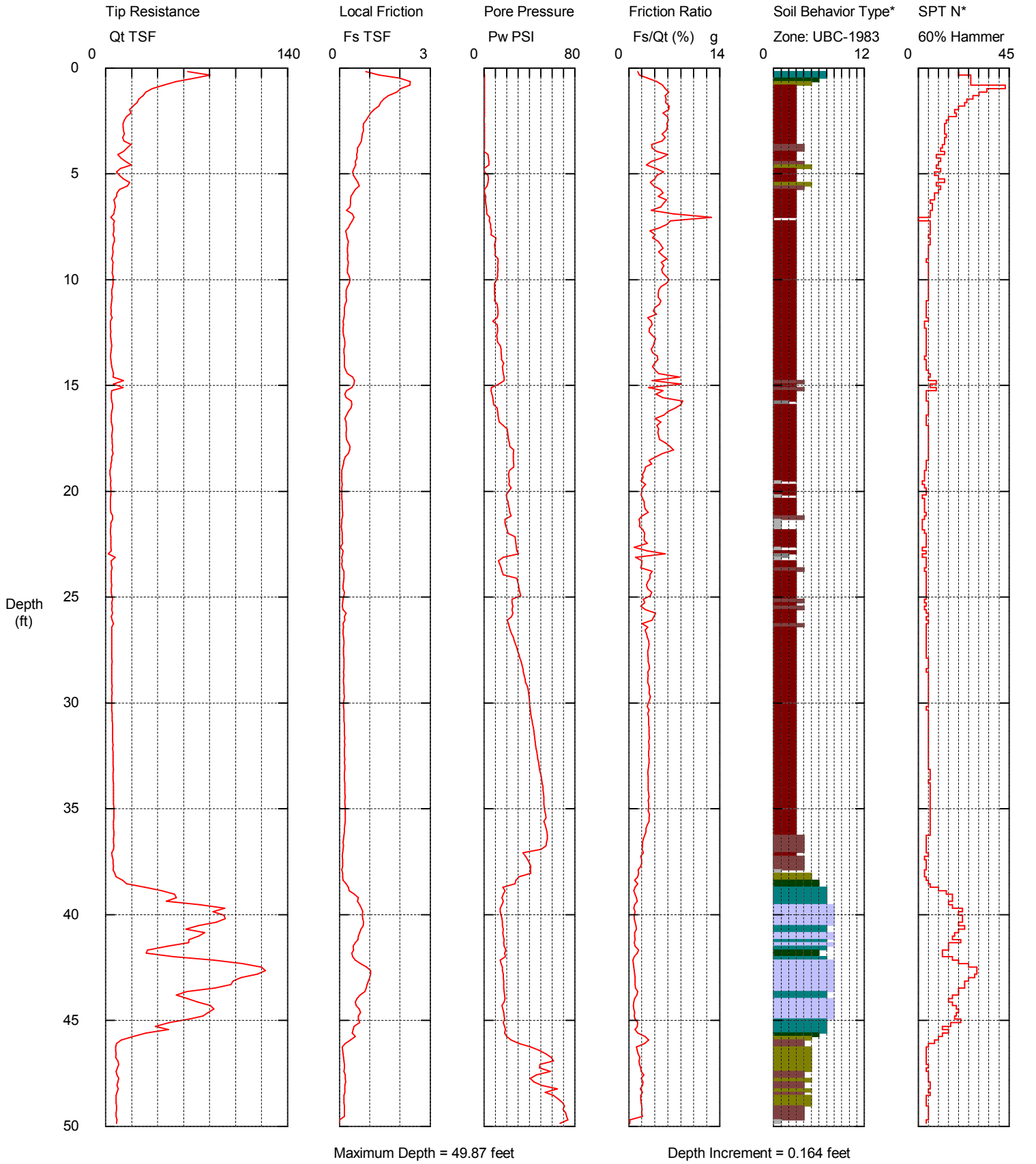
*Soil behavior type and SPT based on data from UBC-1983

Phase I CPT 03

Southern Earth Sciences, Inc.

Operator: WRP
Sounding: CPT-03
Cone Used: DDG0899

CPT Date/Time: 11/10/2005 12:30:00 PM
Location: 17th Street Canal Levee
Job Number: B05-157



Maximum Depth = 49.87 feet

Depth Increment = 0.164 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

A.IX

GPS Coordinates
N30 00.945' W90 07.289'

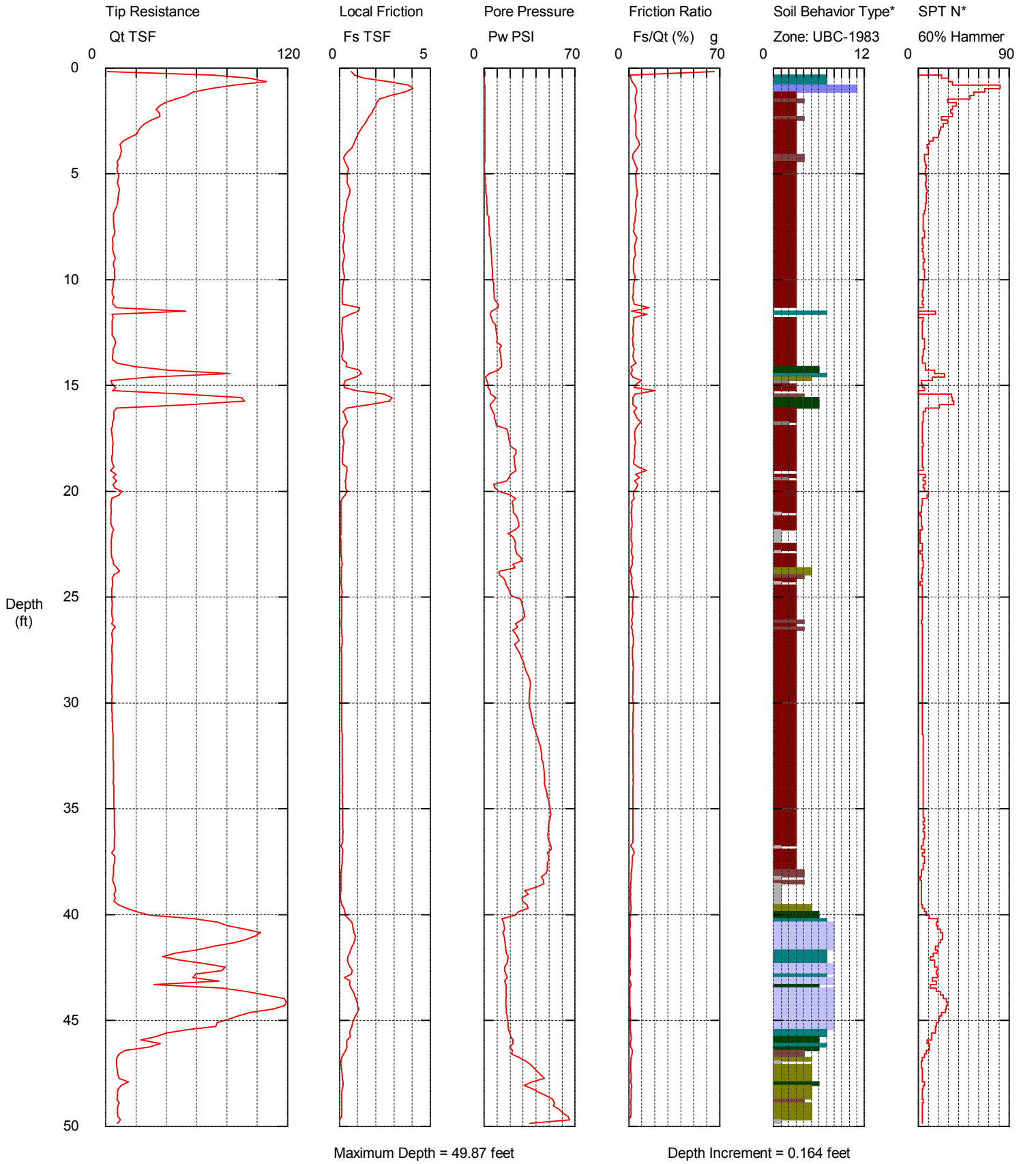
*Soil behavior type and SPT based on data from UBC-1983

Phase I CPT 04

Southern Earth Sciences, Inc.

Operator: WRP
Sounding: CPT-04
Cone Used: DDG0899

CPT Date/Time: 11/10/2005 3:19:01 PM
Location: 17th Street Canal Levee
Job Number: B05-157



Maximum Depth = 49.87 feet

Depth Increment = 0.164 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

A.IX

GPS Coordinates
N30 00.967' W90 07.286'

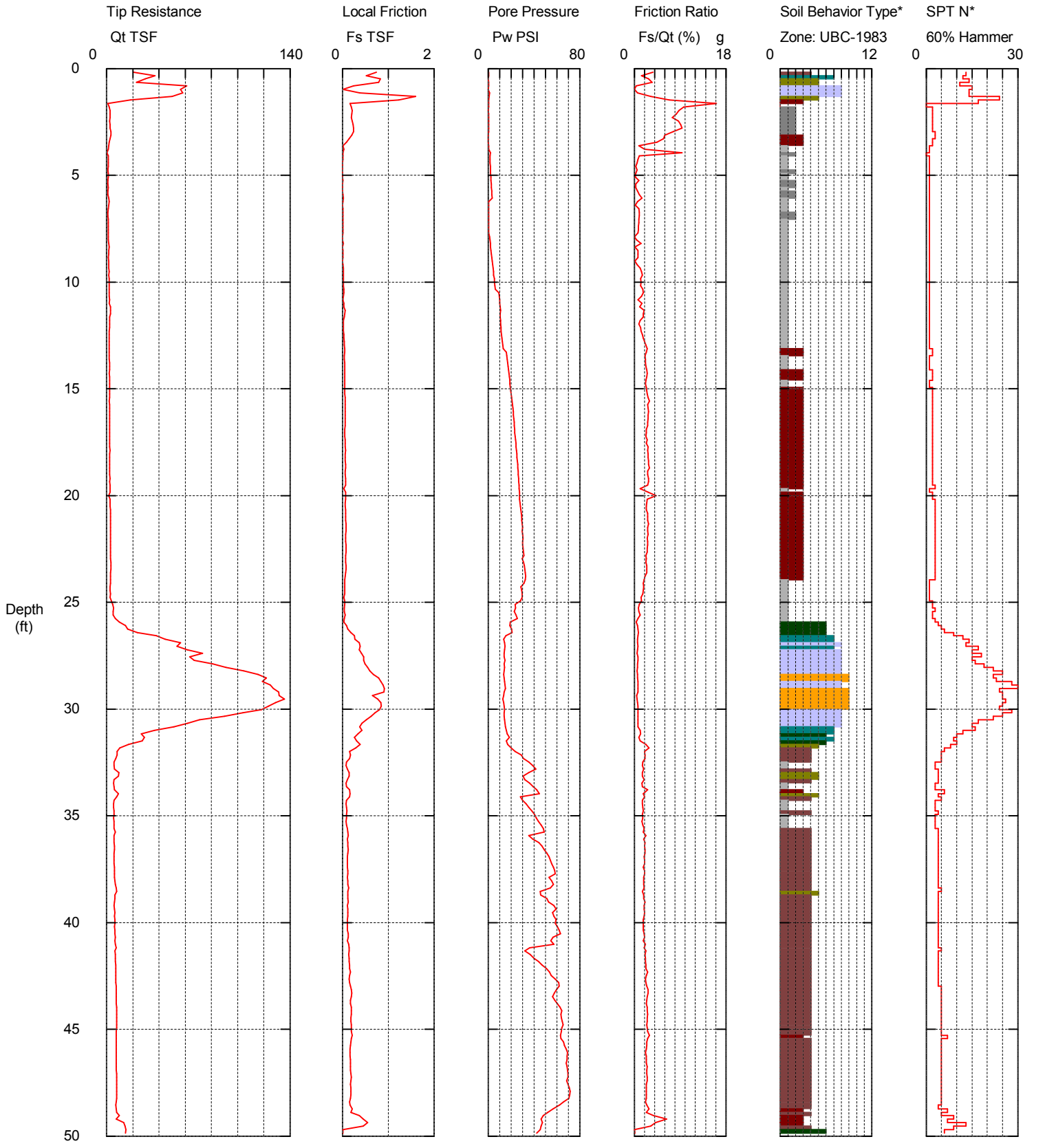
*Soil behavior type and SPT based on data from UBC-1983

Phase I CPT 05

Southern Earth Sciences, Inc.

Operator: WRP
Sounding: CPT-05
Cone Used: DDG0899

CPT Date/Time: 11/11/2005 11:35:46 AM
Location: 17th Street Canal Levee
Job Number: B05-157



Maximum Depth = 49.87 feet

Depth Increment = 0.164 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

A.IX

GPS Coordinates
N30 00.970' W90 07.244'

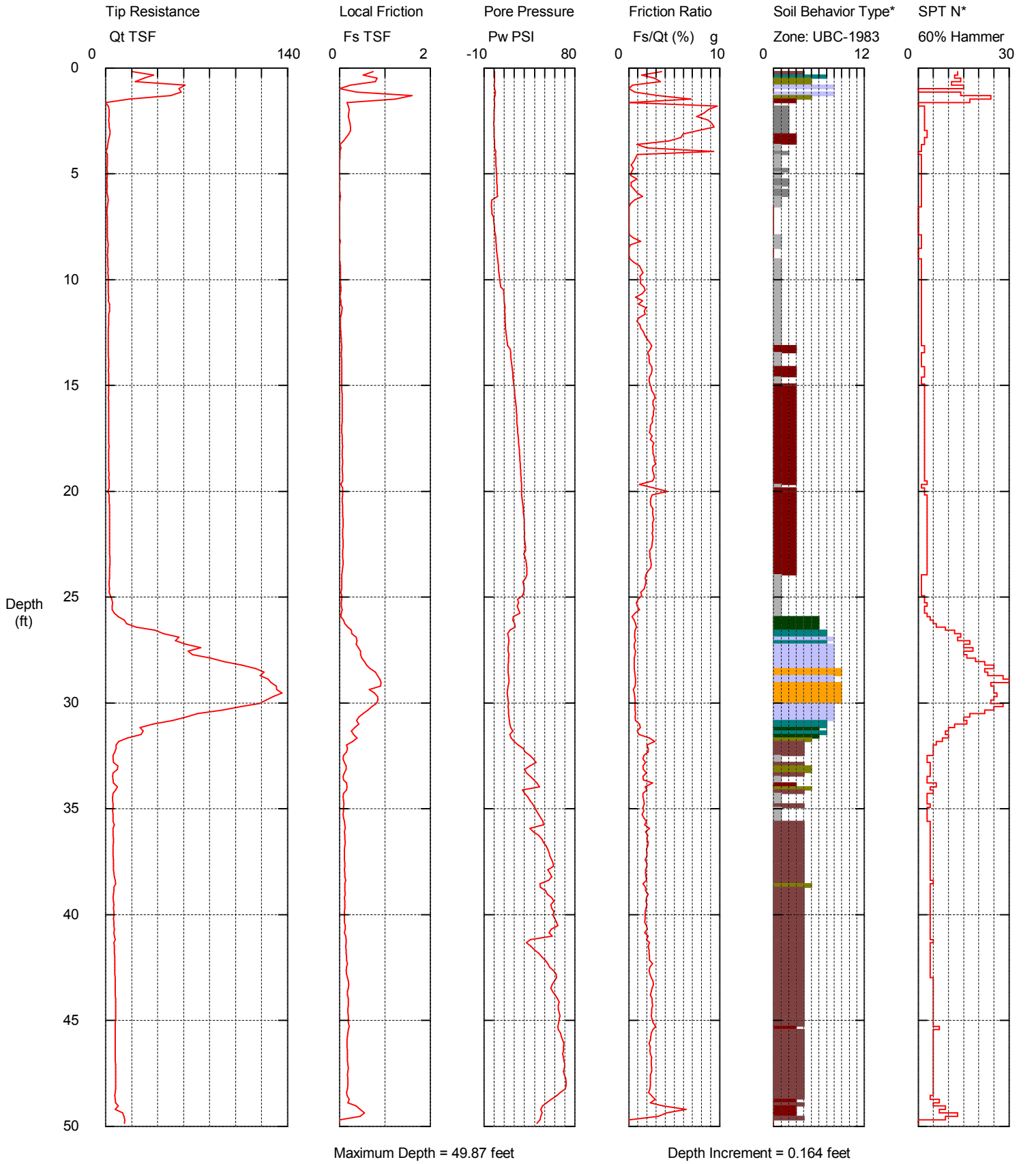
*Soil behavior type and SPT based on data from UBC-1983

Phase I CPT 05A

Southern Earth Sciences, Inc.

Operator: WRP
Sounding: CPT-05A
Cone Used: DDG0899

CPT Date/Time: 11/11/2005 11:35:46 AM
Location: 17TH STREET
Job Number: B05-157



Maximum Depth = 49.87 feet

Depth Increment = 0.164 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

A.IX

N 30° 00.970'
W 90° 07.244'

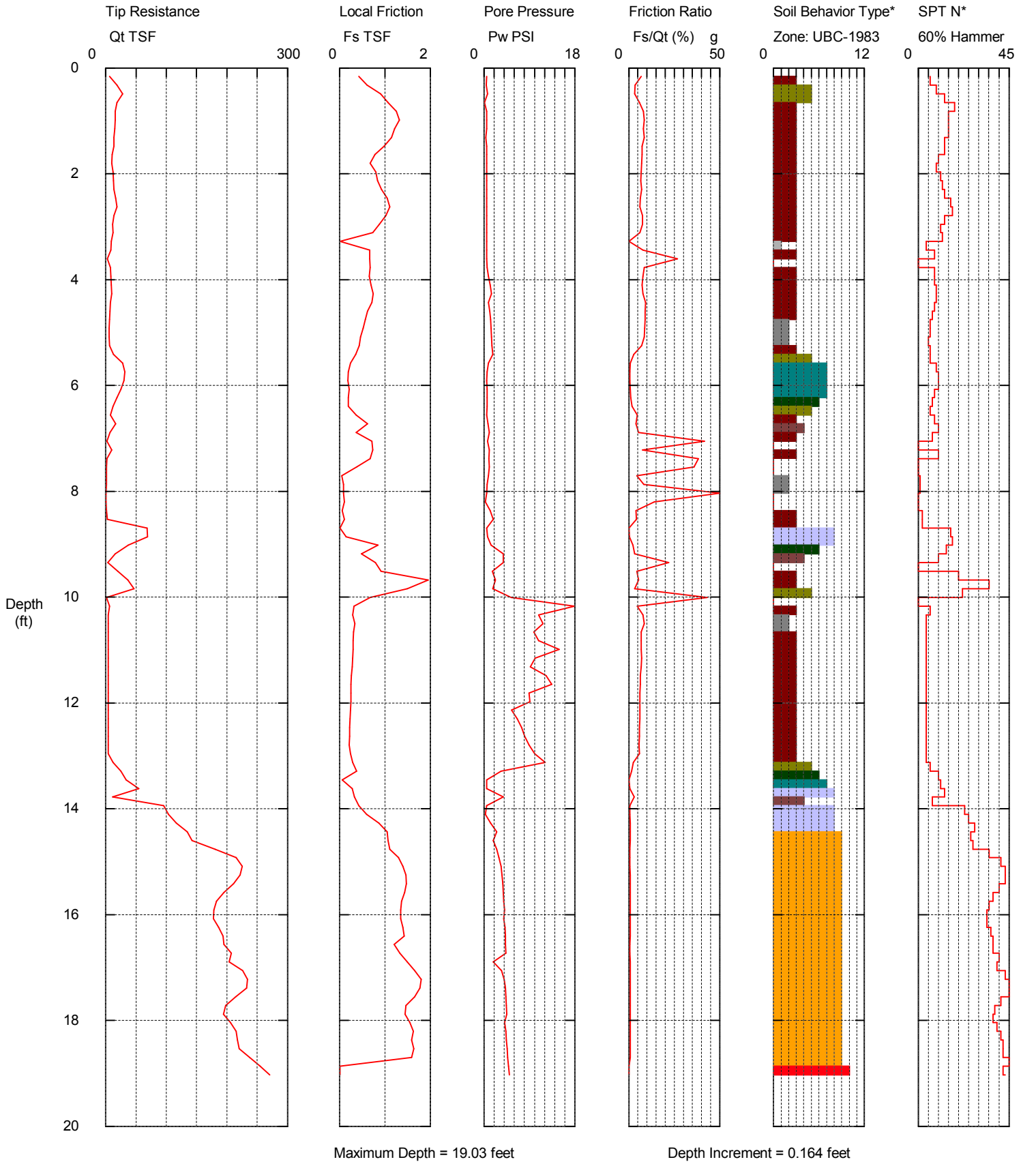
*Soil behavior type and SPT based on data from UBC-1983

Phase I LCPT 01

Southern Earth Sciences, Inc.

Operator: WRP
Sounding: LCPT-01
Cone Used: DDG0899

CPT Date/Time: 11/11/2005 2:44:07 PM
Location: London Avenue Levee @ Maribeau
Job Number: B05-157



Maximum Depth = 19.03 feet

Depth Increment = 0.164 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

A.IX

GPS Coordinates
N30 00.538' W90 04.167'

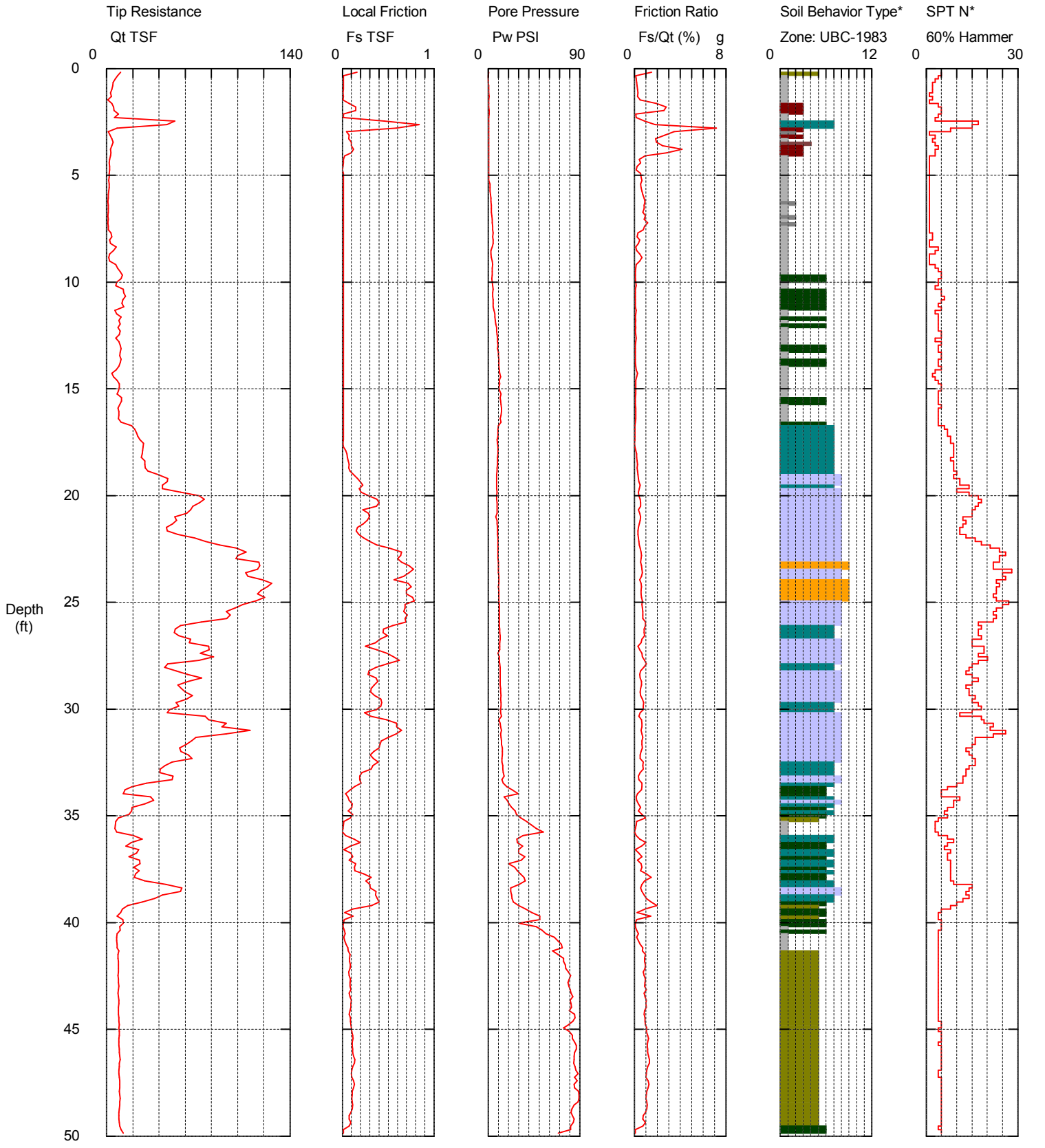
*Soil behavior type and SPT based on data from UBC-1983

Phase I LCPT 02

Southern Earth Sciences, Inc.

Operator: WRP
Sounding: LCPT-02
Cone Used: DDG0899

CPT Date/Time: 11/11/2005 5:07:48 PM
Location: London Avenue Levee @ R.E. Lee
Job Number: B05-157



Maximum Depth = 49.87 feet

Depth Increment = 0.164 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

A.IX

GPS Coordinates
N30 01.236' W90 04.267'

*Soil behavior type and SPT based on data from UBC-1983

CPT1_INT

SOUTHERN EARTH SCIENCES, INC.

CONE PENETRATION TEST DATA INTERPRETATION PROGRAM
==========

NOTE: The soil properties presented herein were developed using generally accepted interpretation methods as outlined below. These properties are presented for information purposes only. We highly recommend the user fully understand the capabilities of the CPT test before using these values for design purposes.

SUMMARY SHEET

'a' for calculating Qt:	0.800
Value for Water Table (in m):	1.500
Valid Zone Classification based on:	Rf
Missing unit weight to start depth:	18.860
Method for calculating Su:	Nkt
Define Zone 6 for Sand Parameters?	YES
Method for Friction Angle:	Kulhawy and Mayne
Vertical Flow Gradient, i (- up):	+0.000

Soil Behavior Type Zone Numbers
For Rf Zone & Bq Zone Classification

Zone #1=Sensitive fine grained	Zone #7 =Sand with some Silt
Zone #2=Organic material	Zone #8 =Fine sand
Zone #3=Clay	Zone #9 =Sand
Zone #4=Silty clay	Zone #10=Gravelly sand
Zone #5=Clayey silt	Zone #11=Very stiff fine grained *
Zone #6=Silty sand	Zone #12=Sand to clayey sand *

* Overconsolidated and/or cemented

NOTE:

For soil classification, Rf values > 8 are assumed to be 8.

(Note: --- means Out Of Range)

METHODS OF EVALUATING SOIL PROPERTIES

SBT = Soil Behavior Type - Robertson and Campanella 1986
Soil Groups 1-12 - as described below

Su - Undrained Shear Strength = (q_tAvg - sgv0Avg)/Nkt;

q_t - Cone tip resistance q_c , corrected for pore water effects
sgv0= Total Vertical Stress
Nkt - User selected empirical factor, usually between 12-18. Value used reported in output.
Su valid for SBT 1:5

Phi - Internal Friction Angle = 17.6 + (11*log10(q_cAvg./sqrt(sgv0pAvg))); (Kulhway and Mayne , 1990)
sgv0p = Vertical effective stress

Phi valid for SBT 6:9

Calculation of N60 - Equivalent SPT Blow Count

SBT	Soil Type	qc/N
1,	Sensitive fine grained	2
2,	Organic Material	1
3,	Clay	1
4,	Silty CLay to clay	1.5
5,	Clayey Silt to silty clay	2
6,	Sandy Silt to clayey sand	2.5
7,	Sitly Sand to sandy silt	3
8,	Sand to silty sand	4
9,	Sand	5
10,	Gravelly Sand to sand	6
11,	Very stiff fine grained *	1
12,	Sand to clayey sand *	2

* Overconsolidated to cemented

CPT1_INT

"
"
"Total Depth: 86.4501 (ft)
"Sounding ID: LEVEE.CPT-01
"Nkt: 18.00
"

"depth (feet)	q_tAvg (tsf)	f_sAvg (tsf)	u_2Avg (psi)	N60 (n/a)	SBT (n/a)	Phi (deg)	Su (psf)
0.9	58.580	1.739	0.036	23	6	40	---
1.1	49.890	1.740	0.036	25	5	---	5536
2.1	26.092	1.111	0.036	17	4	---	2886
3.0	16.483	0.746	0.036	11	4	---	1812
4.0	8.745	0.659	0.036	9	3	---	946
5.0	8.506	0.621	0.036	8	3	---	914
6.0	9.799	0.505	0.032	10	3	---	1051
7.0	10.465	0.374	0.036	7	4	---	1119
8.0	7.642	0.433	0.024	8	3	---	799
8.9	7.827	0.383	0.033	8	3	---	814
9.9	5.843	0.277	0.071	6	3	---	587
11.1	6.251	0.233	0.163	6	3	---	625
12.1	4.419	0.190	0.243	4	3	---	416
13.0	4.465	0.171	0.323	4	3	---	415
14.0	7.914	0.524	0.313	8	3	---	792
15.0	5.462	0.377	0.311	5	3	---	513
16.0	5.624	0.404	0.463	6	3	---	525
17.0	4.690	0.262	0.622	5	3	---	415
18.0	4.837	0.274	0.769	5	3	---	426
18.9	3.501	0.105	0.883	3	3	---	271
19.9	3.178	0.081	0.980	3	3	---	229
21.1	3.272	0.083	1.179	3	3	---	233
22.1	3.367	0.095	1.312	3	3	---	237
23.0	3.336	0.103	1.448	3	3	---	228
24.0	3.507	0.125	1.672	3	3	---	241
25.0	3.733	0.126	1.840	3	3	---	260
26.0	4.062	0.160	1.857	4	3	---	290
27.0	4.059	0.148	2.044	4	3	---	284
28.0	4.167	0.147	2.217	4	3	---	290
29.0	4.262	0.157	2.366	4	3	---	294
29.9	4.350	0.155	2.520	4	3	---	298
30.9	4.498	0.162	2.623	4	3	---	308
32.1	4.817	0.170	2.713	4	3	---	336
33.1	5.139	0.178	2.871	5	3	---	366
34.0	5.378	0.184	3.009	5	3	---	387
35.0	5.603	0.188	3.126	5	3	---	405
36.0	5.964	0.149	2.710	5	3	---	439
37.0	9.355	0.141	1.612	5	5	---	810
38.0	55.935	0.360	1.210	14	8	37	---
39.0	97.875	0.741	1.010	20	9	40	---
39.9	98.031	0.769	1.013	24	8	40	---
40.9	55.097	0.447	1.080	14	8	37	---
42.1	70.734	0.561	1.142	18	8	38	---
43.1	52.433	0.458	1.153	13	8	36	---
44.0	23.002	0.327	1.257	9	6	32	---
45.0	7.554	0.136	1.747	5	4	---	557
46.0	7.204	0.132	2.286	4	4	---	512
47.0	7.419	0.145	2.683	5	4	---	529
48.0	7.078	0.141	3.097	4	4	---	485
49.0	7.191	0.172	3.684	4	4	---	491
50.0	7.347	0.168	4.147	4	4	---	502
50.9	7.208	0.167	3.707	4	4	---	481
51.9	7.112	0.157	3.086	4	4	---	464
53.1	7.219	0.173	3.817	4	4	---	468
54.1	7.792	0.208	4.338	5	4	---	526
55.0	7.864	0.210	4.997	5	4	---	528
56.0	8.139	0.237	5.270	5	4	---	552
57.0	8.649	0.261	5.388	5	4	---	602
58.0	8.343	0.243	5.465	5	4	---	562
59.0	8.277	0.227	5.515	5	4	---	549
60.0	8.333	0.222	5.707	5	4	---	549
60.9	8.617	0.218	5.790	5	4	---	574
61.9	8.421	0.186	5.722	5	4	---	546
63.1	15.553	0.324	4.259	7	5	---	1331
64.1	26.334	0.806	0.466	13	5	---	2523
65.0	29.934	0.993	0.459	15	5	---	2916
66.0	43.817	1.778	0.513	22	5	---	4453
67.0	66.503	1.635	0.551	27	6	36	---
68.0	78.331	1.176	0.597	26	7	37	---

CPT1_INT

69.0	72.747	1.307	0.642	24	7	37	---
70.0	91.036	1.630	0.677	30	7	38	---
70.9	85.050	1.591	0.758	28	7	37	---
71.9	76.446	1.570	0.754	25	7	37	---
73.1	35.975	1.067	0.973	18	5	---	3535
74.1	34.251	0.956	1.103	14	6	33	---
75.0	47.248	1.276	1.015	19	6	34	---
76.0	25.409	0.878	1.180	13	5	---	2343
77.0	26.489	1.007	1.306	13	5	---	2456
78.0	26.230	0.943	1.408	13	5	---	2421
79.0	29.213	0.996	1.498	14	5	---	2746
80.0	44.511	1.480	1.528	22	5	---	4440
81.0	28.872	0.816	1.554	11	6	32	---
81.9	25.575	0.605	1.921	10	6	31	---
82.9	61.357	2.068	2.781	30	5	---	6293
84.1	85.237	1.947	2.535	34	6	37	---
85.1	24.906	0.821	1.946	12	5	---	2229
85.8	25.348	0.609	2.312	10	6	31	---

CPT2_INT

SOUTHERN EARTH SCIENCES, INC.

CONE PENETRATION TEST DATA INTERPRETATION PROGRAM
==========

NOTE: The soil properties presented herein were developed using generally accepted interpretation methods as outlined below. These properties are presented for information purposes only. We highly recommend the user fully understand the capabilities of the CPT test before using these values for design purposes.

SUMMARY SHEET

'a' for calculating Qt:	0.800
Value for Water Table (in m):	1.500
Valid Zone Classification based on:	Rf
Missing unit weight to start depth:	18.860
Method for calculating Su:	Nkt
Define Zone 6 for Sand Parameters?	YES
Method for Friction Angle:	Kulhawy and Mayne
Vertical Flow Gradient, i (- up):	+0.000

Soil Behavior Type Zone Numbers
For Rf Zone & Bq Zone Classification

Zone #1=Sensitive fine grained	Zone #7 =Sand with some Silt
Zone #2=Organic material	Zone #8 =Fine sand
Zone #3=Clay	Zone #9 =Sand
Zone #4=Silty clay	Zone #10=Gravelly sand
Zone #5=Clayey silt	Zone #11=Very stiff fine grained *
Zone #6=Silty sand	Zone #12=Sand to clayey sand *

* Overconsolidated and/or cemented

NOTE:

For soil classification, Rf values > 8 are assumed to be 8.

(Note: --- means Out Of Range)

METHODS OF EVALUATING SOIL PROPERTIES

SBT = Soil Behavior Type - Robertson and Campanella 1986
Soil Groups 1-12 - as described below

Su - Undrained Shear Strength = (q_tAvg - sgv0Avg)/Nkt;

q_t - Cone tip resistance q_c , corrected for pore water effects
sgv0= Total Vertical Stress
Nkt - User selected empirical factor, usually between 12-18. Value used reported in output.
Su valid for SBT 1:5

Phi - Internal Friction Angle = 17.6 + (11*log10(q_cAvg./sqrt(sgv0pAvg))); (Kulhway and Mayne , 1990)
sgv0p = Vertical effective stress

Phi valid for SBT 6:9

Calculation of N60 - Equivalent SPT Blow Count

SBT	Soil Type	qc/N
1,	Sensitive fine grained	2
2,	Organic Material	1
3,	Clay	1
4,	Silty CLay to clay	1.5
5,	Clayey Silt to silty clay	2
6,	Sandy Silt to clayey sand	2.5
7,	Sitly Sand to sandy silt	3
8,	Sand to silty sand	4
9,	Sand	5
10,	Gravelly Sand to sand	6
11,	Very stiff fine grained *	1
12,	Sand to clayey sand *	2

* Overconsolidated to cemented

CPT2_INT

"
"
"Total Depth: 50.0328 (ft)
"Sounding ID: LEVEE.CPT-02
"Nkt: 18.00
"

"depth (feet)	q_tAvg (tsf)	f_sAvg (tsf)	u_2Avg (psi)	N60 (n/a)	SBT (n/a)	Phi (deg)	Su (psf)
0.9	53.548	2.094	0.024	27	5	---	5944
1.1	43.871	2.054	0.029	29	4	---	4868
2.1	24.375	1.407	0.036	24	3	---	2696
3.0	21.047	1.150	0.036	21	3	---	2320
4.0	14.585	0.811	0.036	15	3	---	1596
5.0	23.754	0.674	0.036	16	4	---	2608
6.0	32.312	0.655	0.036	13	6	39	---
7.0	18.453	0.499	0.034	9	5	---	2007
8.0	13.942	0.434	0.052	9	4	---	1499
8.9	7.716	0.314	0.065	8	3	---	802
9.9	5.588	0.188	0.025	6	3	---	559
11.1	5.019	0.193	0.036	5	3	---	489
12.1	4.053	0.131	0.046	4	3	---	375
13.0	4.714	0.127	0.054	5	3	---	443
14.0	6.989	0.357	0.072	7	3	---	689
15.0	7.470	0.466	0.111	7	3	---	737
16.0	6.559	0.433	0.132	7	3	---	629
17.0	4.961	0.198	0.150	5	3	---	446
18.0	5.020	0.239	0.179	5	3	---	446
18.9	4.198	0.166	0.190	4	3	---	349
19.9	3.984	0.085	0.196	4	2	---	319
21.1	3.021	0.031	0.228	1	1	---	205
22.1	3.222	0.033	0.243	2	1	---	221
23.0	3.249	0.044	0.260	2	1	---	218
24.0	3.471	0.059	0.304	2	1	---	237
25.0	3.818	0.075	0.320	2	1	---	269
26.0	4.093	0.079	0.340	2	1	---	293
27.0	4.141	0.082	0.370	4	2	---	293
28.0	4.124	0.084	0.394	4	2	---	285
29.0	4.206	0.084	0.410	4	2	---	288
29.9	4.348	0.090	0.430	4	3	---	297
30.9	4.605	0.095	0.478	5	3	---	320
32.1	4.928	0.108	0.503	5	3	---	349
33.1	5.124	0.111	0.527	3	4	---	364
34.0	5.383	0.115	0.567	4	4	---	387
35.0	5.604	0.125	0.590	4	4	---	405
36.0	5.895	0.136	0.614	4	4	---	431
37.0	5.884	0.108	0.658	6	3	---	424
38.0	6.077	0.068	0.636	3	1	---	439
39.0	17.078	0.157	0.653	11	4	---	1655
39.9	55.924	0.474	0.647	14	8	37	---
40.9	59.364	0.454	0.657	15	8	37	---
42.1	44.303	0.385	0.684	15	7	36	---
43.1	67.865	0.460	0.721	17	8	38	---
44.0	59.664	0.466	0.761	15	8	37	---
45.0	34.844	0.409	0.785	12	7	34	---
46.0	8.700	0.185	0.850	4	5	---	678
47.0	11.548	0.217	0.923	6	5	---	988
48.0	8.447	0.139	0.985	8	3	---	637
49.0	7.269	0.090	1.100	4	1	---	500
49.4	7.450	0.080	1.175	4	1	---	518

CPT3_INT

SOUTHERN EARTH SCIENCES, INC.

CONE PENETRATION TEST DATA INTERPRETATION PROGRAM
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NOTE: The soil properties presented herein were developed using generally accepted interpretation methods as outlined below. These properties are presented for information purposes only. We highly recommend the user fully understand the capabilities of the CPT test before using these values for design purposes.

SUMMARY SHEET

'a' for calculating Qt:	0.800
Value for Water Table (in m):	1.500
Valid Zone Classification based on:	Rf
Missing unit weight to start depth:	18.860
Method for calculating Su:	Nkt
Define Zone 6 for Sand Parameters?	YES
Method for Friction Angle:	Kulhawy and Mayne
Vertical Flow Gradient, i (- up):	+0.000

Soil Behavior Type Zone Numbers
For Rf Zone & Bq Zone Classification

Zone #1=Sensitive fine grained	Zone #7 =Sand with some Silt
Zone #2=Organic material	Zone #8 =Fine sand
Zone #3=Clay	Zone #9 =Sand
Zone #4=Silty clay	Zone #10=Gravelly sand
Zone #5=Clayey silt	Zone #11=Very stiff fine grained *
Zone #6=Silty sand	Zone #12=Sand to clayey sand *

* Overconsolidated and/or cemented

NOTE:

For soil classification, Rf values > 8 are assumed to be 8.

(Note: --- means Out Of Range)

METHODS OF EVALUATING SOIL PROPERTIES

SBT = Soil Behavior Type - Robertson and Campanella 1986
Soil Groups 1-12 - as described below

Su - Undrained Shear Strength = (q_tAvg - sgv0Avg)/Nkt;

q_t - Cone tip resistance q_c , corrected for pore water effects
sgv0= Total Vertical Stress
Nkt - User selected empirical factor, usually between 12-18. Value used reported in output.
Su valid for SBT 1:5

Phi - Internal Friction Angle = 17.6 + (11*log10(q_cAvg./sqrt(sgv0pAvg))); (Kulhway and Mayne , 1990)
sgv0p = Vertical effective stress

Phi valid for SBT 6:9

Calculation of N60 - Equivalent SPT Blow Count

SBT	Soil Type	qc/N
1,	Sensitive fine grained	2
2,	Organic Material	1
3,	Clay	1
4,	Silty CLay to clay	1.5
5,	Clayey Silt to silty clay	2
6,	Sandy Silt to clayey sand	2.5
7,	Sitly Sand to sandy silt	3
8,	Sand to silty sand	4
9,	Sand	5
10,	Gravelly Sand to sand	6
11,	Very stiff fine grained *	1
12,	Sand to clayey sand *	2

* Overconsolidated to cemented

CPT3_INT

"
"
"Total Depth: 49.8688 (ft)
"Sounding ID: LEVEE.CPT-03
"Nkt: 18.00
"

"depth (feet)	q_tAvg (tsf)	f_sAvg (tsf)	u_2Avg (psi)	N60 (n/a)	SBT (n/a)	Phi (deg)	Su (psf)
0.9	43.849	1.885	0.036	29	4	---	4866
1.1	36.670	1.885	0.036	37	3	---	4068
2.1	18.997	1.115	0.034	19	3	---	2098
3.0	13.951	0.763	0.034	14	3	---	1531
4.0	15.098	0.591	0.154	10	4	---	1653
5.0	14.364	0.527	0.215	10	4	---	1565
6.0	9.766	0.451	0.106	10	3	---	1048
7.0	6.458	0.371	0.259	6	3	---	674
8.0	6.046	0.268	0.570	6	3	---	623
8.9	5.381	0.275	0.784	5	3	---	543
9.9	5.373	0.288	0.788	5	3	---	536
11.1	4.447	0.198	0.768	4	3	---	426
12.1	3.831	0.135	0.844	4	3	---	351
13.0	4.033	0.154	0.953	4	3	---	367
14.0	4.595	0.193	1.172	4	3	---	424
15.0	6.623	0.344	0.823	6	3	---	643
16.0	4.570	0.279	0.713	4	3	---	409
17.0	4.749	0.224	1.259	4	3	---	423
18.0	4.995	0.261	1.722	5	3	---	444
18.9	3.858	0.102	1.731	4	3	---	312
19.9	3.729	0.079	1.543	3	2	---	291
21.1	3.984	0.086	1.506	4	2	---	312
22.1	3.813	0.081	1.723	3	2	---	287
23.0	4.213	0.092	1.570	4	2	---	326
24.0	4.392	0.126	1.631	4	3	---	340
25.0	4.651	0.126	2.004	4	3	---	362
26.0	4.880	0.141	1.687	5	3	---	382
27.0	4.646	0.134	1.869	4	3	---	350
28.0	4.725	0.142	2.304	4	3	---	352
29.0	4.770	0.145	2.630	4	3	---	351
29.9	4.766	0.148	2.870	4	3	---	345
30.9	5.176	0.158	3.052	5	3	---	384
32.1	5.486	0.170	3.288	5	3	---	412
33.1	5.709	0.172	3.509	5	3	---	430
34.0	5.894	0.177	3.728	5	3	---	445
35.0	6.141	0.188	3.823	5	3	---	466
36.0	6.156	0.163	3.945	5	3	---	462
37.0	5.721	0.119	3.295	3	4	---	407
38.0	7.508	0.110	2.622	3	5	---	599
39.0	40.656	0.424	1.342	13	7	35	---
39.9	82.465	0.745	1.111	21	8	39	---
40.9	68.130	0.655	1.215	17	8	38	---
42.1	73.425	0.582	1.222	18	8	38	---
43.1	103.327	0.947	1.224	26	8	40	---
44.0	73.226	0.665	1.250	18	8	38	---
45.0	59.016	0.581	1.272	15	8	37	---
46.0	14.736	0.276	2.197	7	5	---	1349
47.0	8.410	0.145	3.823	4	5	---	640
48.0	8.787	0.179	3.760	4	5	---	676
49.0	8.111	0.159	4.896	5	4	---	594
49.4	8.184	0.130	5.092	7	3	---	600

CPT4_INT

SOUTHERN EARTH SCIENCES, INC.

CONE PENETRATION TEST DATA INTERPRETATION PROGRAM
==========

NOTE: The soil properties presented herein were developed using generally accepted interpretation methods as outlined below. These properties are presented for information purposes only. We highly recommend the user fully understand the capabilities of the CPT test before using these values for design purposes.

SUMMARY SHEET

'a' for calculating Qt:	0.800
Value for Water Table (in m):	1.500
Valid Zone Classification based on:	Rf
Missing unit weight to start depth:	18.860
Method for calculating Su:	Nkt
Define Zone 6 for Sand Parameters?	YES
Method for Friction Angle:	Kulhawy and Mayne
Vertical Flow Gradient, i (- up):	+0.000

Soil Behavior Type Zone Numbers
For Rf Zone & Bq Zone Classification

Zone #1=Sensitive fine grained	Zone #7 =Sand with some Silt
Zone #2=Organic material	Zone #8 =Fine sand
Zone #3=Clay	Zone #9 =Sand
Zone #4=Silty clay	Zone #10=Gravelly sand
Zone #5=Clayey silt	Zone #11=Very stiff fine grained *
Zone #6=Silty sand	Zone #12=Sand to clayey sand *

* Overconsolidated and/or cemented

NOTE:

For soil classification, Rf values > 8 are assumed to be 8.

(Note: --- means Out Of Range)

METHODS OF EVALUATING SOIL PROPERTIES

SBT = Soil Behavior Type - Robertson and Campanella 1986
Soil Groups 1-12 - as described below

Su - Undrained Shear Strength = (q_tAvg - sgv0Avg)/Nkt;

q_t - Cone tip resistance q_c , corrected for pore water effects
sgv0= Total Vertical Stress
Nkt - User selected empirical factor, usually between 12-18. Value used reported in output.
Su valid for SBT 1:5

Phi - Internal Friction Angle = 17.6 + (11*log10(q_cAvg./sqrt(sgv0pAvg))); (Kulhway and Mayne , 1990)
sgv0p = Vertical effective stress

Phi valid for SBT 6:9

Calculation of N60 - Equivalent SPT Blow Count

SBT	Soil Type	qc/N
1,	Sensitive fine grained	2
2,	Organic Material	1
3,	Clay	1
4,	Silty CLay to clay	1.5
5,	Clayey Silt to silty clay	2
6,	Sandy Silt to clayey sand	2.5
7,	Sitly Sand to sandy silt	3
8,	Sand to silty sand	4
9,	Sand	5
10,	Gravelly Sand to sand	6
11,	Very stiff fine grained *	1
12,	Sand to clayey sand *	2

* Overconsolidated to cemented

CPT4_INT

"
"
"Total Depth: 49.8688 (ft)
"Sounding ID: LEVEE.CPT-04
"Nkt: 18.00
"

"depth (feet)	q_tAvg (tsf)	f_sAvg (tsf)	u_2Avg (psi)	N60 (n/a)	SBT (n/a)	Phi (deg)	Su (psf)
0.9	71.675	2.747	0.031	24	7	40	---
1.1	67.587	2.862	0.031	27	6	40	---
2.1	34.933	1.807	0.036	35	3	---	3868
3.0	19.703	1.107	0.036	20	3	---	2170
4.0	9.481	0.490	0.036	9	3	---	1028
5.0	7.974	0.439	0.042	8	3	---	855
6.0	8.183	0.468	0.124	8	3	---	872
7.0	5.756	0.287	0.214	6	3	---	596
8.0	5.156	0.230	0.321	5	3	---	523
8.9	5.220	0.215	0.409	5	3	---	524
9.9	5.364	0.202	0.484	5	3	---	534
11.1	5.044	0.378	0.576	5	3	---	492
12.1	4.337	0.227	0.542	4	3	---	407
13.0	5.080	0.169	0.795	5	3	---	484
14.0	17.694	0.517	0.798	12	4	---	1879
15.0	29.650	1.013	0.256	20	4	---	3202
16.0	35.879	1.321	0.479	24	4	---	3888
17.0	4.588	0.260	0.971	4	3	---	405
18.0	4.475	0.199	1.564	4	3	---	387
18.9	4.925	0.300	1.604	5	3	---	431
19.9	5.987	0.312	1.176	6	3	---	543
21.1	3.569	0.080	1.735	3	3	---	267
22.1	3.879	0.085	1.731	4	2	---	296
23.0	4.197	0.099	1.797	4	3	---	325
24.0	5.203	0.105	1.217	3	4	---	430
25.0	4.160	0.123	1.757	4	3	---	308
26.0	4.385	0.115	2.033	4	3	---	327
27.0	4.328	0.127	1.806	4	3	---	315
28.0	4.134	0.122	2.109	4	3	---	287
29.0	4.105	0.128	2.490	4	3	---	278
29.9	4.050	0.130	2.525	4	3	---	266
30.9	4.421	0.136	2.719	4	3	---	301
32.1	4.859	0.154	3.098	4	3	---	342
33.1	5.042	0.159	3.290	4	3	---	357
34.0	5.330	0.165	3.420	5	3	---	382
35.0	5.569	0.170	3.630	5	3	---	403
36.0	5.741	0.181	3.585	5	3	---	416
37.0	5.719	0.165	3.595	5	3	---	408
38.0	5.188	0.108	3.427	5	3	---	342
39.0	5.923	0.082	2.563	3	1	---	418
39.9	33.875	0.361	1.719	11	7	34	---
40.9	87.033	0.773	1.121	22	8	39	---
42.1	61.385	0.572	1.221	15	8	37	---
43.1	70.649	0.625	1.192	18	8	38	---
44.0	106.768	0.905	1.224	27	8	40	---
45.0	77.526	0.781	1.298	19	8	38	---
46.0	29.040	0.457	1.500	11	6	33	---
47.0	7.796	0.136	2.349	5	4	---	573
48.0	8.488	0.151	2.970	4	5	---	644
49.0	7.789	0.130	3.914	4	5	---	560
49.4	8.042	0.104	4.184	5	4	---	585

CPT5_INT

SOUTHERN EARTH SCIENCES, INC.

CONE PENETRATION TEST DATA INTERPRETATION PROGRAM
==========

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SUMMARY SHEET

'a' for calculating Qt:	0.800
Value for Water Table (in m):	1.500
Valid Zone Classification based on:	Rf
Missing unit weight to start depth:	18.860
Method for calculating Su:	Nkt
Define Zone 6 for Sand Parameters?	YES
Method for Friction Angle:	Kulhawy and Mayne
Vertical Flow Gradient, i (- up):	+0.000

Soil Behavior Type Zone Numbers
For Rf Zone & Bq Zone Classification

Zone #1=Sensitive fine grained	Zone #7 =Sand with some Silt
Zone #2=Organic material	Zone #8 =Fine sand
Zone #3=Clay	Zone #9 =Sand
Zone #4=Silty clay	Zone #10=Gravelly sand
Zone #5=Clayey silt	Zone #11=Very stiff fine grained *
Zone #6=Silty sand	Zone #12=Sand to clayey sand *

* Overconsolidated and/or cemented

NOTE:

For soil classification, Rf values > 8 are assumed to be 8.

(Note: --- means Out Of Range)

METHODS OF EVALUATING SOIL PROPERTIES

SBT = Soil Behavior Type - Robertson and Campanella 1986
Soil Groups 1-12 - as described below

Su - Undrained Shear Strength = (q_tAvg - sgv0Avg)/Nkt;

q_t - Cone tip resistance q_c , corrected for pore water effects
sgv0= Total Vertical Stress
Nkt - User selected empirical factor, usually between 12-18. Value used reported in output.
Su valid for SBT 1:5

Phi - Internal Friction Angle = 17.6 + (11*log10(q_cAvg./sqrt(sgv0pAvg))); (Kulhway and Mayne , 1990)
sgv0p = Vertical effective stress

Phi valid for SBT 6:9

Calculation of N60 - Equivalent SPT Blow Count

SBT	Soil Type	qc/N
1,	Sensitive fine grained	2
2,	Organic Material	1
3,	Clay	1
4,	Silty CLay to clay	1.5
5,	Clayey Silt to silty clay	2
6,	Sandy Silt to clayey sand	2.5
7,	Sitly Sand to sandy silt	3
8,	Sand to silty sand	4
9,	Sand	5
10,	Gravelly Sand to sand	6
11,	Very stiff fine grained *	1
12,	Sand to clayey sand *	2

* Overconsolidated to cemented

CPT5_INT

"
"
"Total Depth: 49.8688 (ft)
"Sounding ID: LEVEE.CPT-05
"Nkt: 18.00
"

depth (feet)	q_tAvg (tsf)	f_sAvg (tsf)	u_2Avg (psi)	N60 (n/a)	SBT (n/a)	Phi (deg)	Su (psf)
0.9	42.331	0.666	0.019	14	7	40	---
1.1	39.194	0.608	0.021	16	6	40	---
2.1	2.423	0.209	-0.005	2	2	---	257
3.0	2.546	0.186	-0.004	3	2	---	266
4.0	1.427	0.015	0.082	1	1	---	136
5.0	1.171	0.004	0.156	1	1	---	102
6.0	1.266	0.006	0.080	1	1	---	108
7.0	1.193	0.000	-0.094	1	1	---	94
8.0	1.451	0.002	0.086	1	1	---	117
8.9	1.585	0.005	0.225	1	1	---	126
9.9	1.854	0.026	0.385	1	1	---	149
11.1	2.400	0.035	0.727	1	1	---	203
12.1	2.176	0.030	0.791	1	1	---	172
13.0	2.001	0.042	0.984	1	1	---	147
14.0	2.126	0.049	1.258	2	2	---	154
15.0	2.129	0.053	1.405	2	2	---	149
16.0	2.181	0.059	1.551	2	3	---	148
17.0	2.257	0.056	1.656	2	3	---	151
18.0	2.370	0.063	1.779	2	3	---	157
18.9	2.429	0.067	1.878	2	3	---	157
19.9	2.612	0.068	1.956	2	3	---	172
21.1	2.888	0.076	2.099	2	3	---	196
22.1	2.992	0.077	2.166	3	3	---	201
23.0	3.095	0.075	2.236	3	3	---	206
24.0	2.888	0.057	2.237	1	1	---	177
25.0	4.003	0.048	1.898	2	1	---	295
26.0	12.572	0.091	1.506	8	4	---	1241
27.0	52.944	0.350	1.030	13	8	38	---
28.0	88.759	0.540	1.000	22	8	40	---
29.0	126.087	0.814	1.000	25	9	40	---
29.9	116.202	0.737	0.984	23	9	40	---
30.9	46.118	0.401	1.099	11	8	37	---
32.1	9.293	0.195	1.865	6	4	---	836
33.1	6.220	0.105	2.603	6	3	---	489
34.0	5.930	0.109	2.592	5	2	---	450
35.0	5.462	0.088	2.948	2	1	---	392
36.0	5.764	0.115	3.277	3	4	---	419
37.0	5.795	0.115	3.832	3	4	---	416
38.0	6.390	0.119	4.012	4	4	---	476
39.0	6.366	0.119	3.798	4	4	---	467
39.9	6.188	0.114	4.270	4	4	---	441
40.9	6.524	0.133	3.634	4	4	---	472
42.1	6.770	0.152	3.478	4	4	---	493
43.1	7.173	0.177	4.212	4	4	---	531
44.0	7.496	0.189	4.465	4	4	---	561
45.0	7.479	0.192	4.613	4	4	---	553
46.0	7.363	0.176	4.860	4	4	---	533
47.0	7.511	0.182	4.970	4	4	---	544
48.0	7.601	0.180	4.989	4	4	---	547
49.0	8.756	0.307	3.745	5	4	---	669
49.4	11.842	0.456	3.349	7	4	---	1010

LCPT1_INT

SOUTHERN EARTH SCIENCES, INC.

CONE PENETRATION TEST DATA INTERPRETATION PROGRAM
==========

NOTE: The soil properties presented herein were developed using generally accepted interpretation methods as outlined below. These properties are presented for information purposes only. We highly recommend the user fully understand the capabilities of the CPT test before using these values for design purposes.

SUMMARY SHEET

'a' for calculating Qt:	0.800
Value for Water Table (in m):	1.500
Valid Zone Classification based on:	Rf
Missing unit weight to start depth:	18.860
Method for calculating Su:	Nkt
Define Zone 6 for Sand Parameters?	YES
Method for Friction Angle:	Kulhawy and Mayne
Vertical Flow Gradient, i (- up):	+0.000

Soil Behavior Type Zone Numbers
For Rf Zone & Bq Zone Classification

Zone #1=Sensitive fine grained	Zone #7 =Sand with some Silt
Zone #2=Organic material	Zone #8 =Fine sand
Zone #3=Clay	Zone #9 =Sand
Zone #4=Silty clay	Zone #10=Gravelly sand
Zone #5=Clayey silt	Zone #11=Very stiff fine grained *
Zone #6=Silty sand	Zone #12=Sand to clayey sand *

* Overconsolidated and/or cemented

NOTE:

For soil classification, Rf values > 8 are assumed to be 8.

(Note: --- means Out Of Range)

METHODS OF EVALUATING SOIL PROPERTIES

SBT = Soil Behavior Type - Robertson and Campanella 1986
Soil Groups 1-12 - as described below

Su - Undrained Shear Strength = (q_tAvg - sgv0Avg)/Nkt;

q_t - Cone tip resistance q_c , corrected for pore water effects
sgv0= Total Vertical Stress
Nkt - User selected empirical factor, usually between 12-18. Value used reported in output.
Su valid for SBT 1:5

Phi - Internal Friction Angle = 17.6 + (11*log10(q_cAvg./sqrt(sgv0pAvg))); (Kulhway and Mayne , 1990)
sgv0p = Vertical effective stress

Phi valid for SBT 6:9

Calculation of N60 - Equivalent SPT Blow Count

SBT	Soil Type	qc/N
1,	Sensitive fine grained	2
2,	Organic Material	1
3,	Clay	1
4,	Silty CLay to clay	1.5
5,	Clayey Silt to silty clay	2
6,	Sandy Silt to clayey sand	2.5
7,	Sitly Sand to sandy silt	3
8,	Sand to silty sand	4
9,	Sand	5
10,	Gravelly Sand to sand	6
11,	Very stiff fine grained *	1
12,	Sand to clayey sand *	2

* Overconsolidated to cemented

LCPT1_INT

"
"
"Total Depth: 19.0289 (ft)
"Sounding ID: LEVEE.LCPT-01
"Nkt: 18.00
"

"depth (feet)	q_tAvg (tsf)	f_sAvg (tsf)	u_2Avg (psi)	N60 (n/a)	SBT (n/a)	Phi (deg)	Su (psf)
0.9	16.169	1.096	0.015	16	3	---	1791
1.1	15.348	1.096	0.007	15	3	---	1699
2.1	13.025	0.896	-0.226	13	3	---	1435
3.0	11.866	0.836	-0.273	12	3	---	1300
4.0	8.269	0.683	0.055	8	3	---	894
5.0	7.724	0.493	0.092	8	3	---	828
6.0	21.016	0.235	0.045	8	6	37	---
7.0	6.665	0.521	0.061	7	3	---	697
8.0	0.737	0.140	0.061	1	2	---	34
8.9	24.673	0.407	0.126	12	5	---	2688
9.9	12.537	0.755	0.426	12	3	---	1334
11.1	4.212	0.292	0.817	4	3	---	402
12.1	3.987	0.243	0.627	4	3	---	371
13.0	13.793	0.247	0.424	7	5	---	1455
14.0	89.891	0.594	-0.025	18	9	40	---
15.0	194.373	1.296	0.222	39	9	40	---
16.0	188.545	1.388	0.288	38	9	40	---
17.0	213.251	1.557	0.294	43	9	40	---
18.0	211.920	1.587	0.315	42	9	40	---
18.5	229.091	1.604	0.328	46	9	40	---

LCPT2_INT

SOUTHERN EARTH SCIENCES, INC.

CONE PENETRATION TEST DATA INTERPRETATION PROGRAM
==========

NOTE: The soil properties presented herein were developed using generally accepted interpretation methods as outlined below. These properties are presented for information purposes only. We highly recommend the user fully understand the capabilities of the CPT test before using these values for design purposes.

SUMMARY SHEET

'a' for calculating Qt:	0.800
Value for Water Table (in m):	1.500
Valid Zone Classification based on:	Rf
Missing unit weight to start depth:	18.860
Method for calculating Su:	Nkt
Define Zone 6 for Sand Parameters?	YES
Method for Friction Angle:	Kulhawy and Mayne
Vertical Flow Gradient, i (- up):	+0.000

Soil Behavior Type Zone Numbers
For Rf Zone & Bq Zone Classification

Zone #1=Sensitive fine grained	Zone #7 =Sand with some Silt
Zone #2=Organic material	Zone #8 =Fine sand
Zone #3=Clay	Zone #9 =Sand
Zone #4=Silty clay	Zone #10=Gravelly sand
Zone #5=Clayey silt	Zone #11=Very stiff fine grained *
Zone #6=Silty sand	Zone #12=Sand to clayey sand *

* Overconsolidated and/or cemented

NOTE:

For soil classification, Rf values > 8 are assumed to be 8.

(Note: --- means Out Of Range)

METHODS OF EVALUATING SOIL PROPERTIES

SBT = Soil Behavior Type - Robertson and Campanella 1986
Soil Groups 1-12 - as described below

Su - Undrained Shear Strength = (q_tAvg - sgv0Avg)/Nkt;

q_t - Cone tip resistance q_c , corrected for pore water effects
sgv0= Total Vertical Stress
Nkt - User selected empirical factor, usually between 12-18. Value used reported in output.
Su valid for SBT 1:5

Phi - Internal Friction Angle = 17.6 + (11*log10(q_cAvg./sqrt(sgv0pAvg))); (Kulhway and Mayne , 1990)
sgv0p = Vertical effective stress

Phi valid for SBT 6:9

Calculation of N60 - Equivalent SPT Blow Count

SBT	Soil Type	qc/N
1,	Sensitive fine grained	2
2,	Organic Material	1
3,	Clay	1
4,	Silty CLay to clay	1.5
5,	Clayey Silt to silty clay	2
6,	Sandy Silt to clayey sand	2.5
7,	Sitly Sand to sandy silt	3
8,	Sand to silty sand	4
9,	Sand	5
10,	Gravelly Sand to sand	6
11,	Very stiff fine grained *	1
12,	Sand to clayey sand *	2

* Overconsolidated to cemented

LCPT2_INT

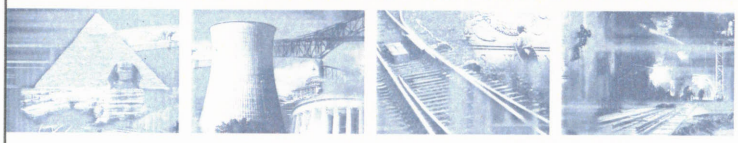
"
"
"Total Depth: 49.8688 (ft)
"Sounding ID: LEVEE.LCPT-02
"Nkt: 18.00
"

"depth (feet)	q_tAvg (tsf)	f_sAvg (tsf)	u_2Avg (psi)	N60 (n/a)	SBT (n/a)	Phi (deg)	Su (psf)
0.9	4.494	0.001	0.012	2	1	---	494
1.1	4.022	0.003	0.012	2	1	---	440
2.1	12.568	0.138	-0.117	13	3	---	1384
3.0	11.554	0.231	-0.258	8	4	---	1265
4.0	2.906	0.054	-0.192	3	2	---	298
5.0	1.896	0.001	-0.041	1	1	---	180
6.0	1.419	0.000	0.166	1	1	---	121
7.0	1.209	0.000	0.257	1	1	---	91
8.0	3.221	0.000	0.307	2	1	---	309
8.9	5.415	0.000	0.253	3	1	---	547
9.9	10.289	0.000	0.285	7	4	---	1082
11.1	11.488	0.000	0.370	6	5	---	1208
12.1	9.365	0.000	0.553	5	1	---	966
13.0	10.154	0.000	0.679	10	2	---	1048
14.0	8.432	0.000	0.770	4	1	---	850
15.0	9.257	0.000	0.826	9	2	---	936
16.0	10.008	0.000	0.887	10	3	---	1013
17.0	20.581	0.000	0.714	7	7	34	---
18.0	27.750	0.039	0.666	9	7	35	---
18.9	36.378	0.117	0.605	9	8	37	---
19.9	58.807	0.276	0.599	15	8	39	---
21.1	54.588	0.245	0.626	14	8	38	---
22.1	71.523	0.294	0.668	18	8	39	---
23.0	107.270	0.663	0.698	21	9	40	---
24.0	116.375	0.712	0.730	23	9	40	---
25.0	107.340	0.707	0.758	21	9	40	---
26.0	69.828	0.590	0.788	17	8	39	---
27.0	68.296	0.430	0.815	17	8	39	---
28.0	62.836	0.425	0.818	16	8	38	---
29.0	61.367	0.346	0.860	15	8	38	---
29.9	59.665	0.385	0.884	15	8	38	---
30.9	82.047	0.531	0.889	20	8	39	---
32.1	58.409	0.370	0.976	15	8	37	---
33.1	42.798	0.240	1.036	14	7	36	---
34.0	24.306	0.094	1.420	8	7	33	---
35.0	13.032	0.065	2.262	6	5	---	1222
36.0	15.484	0.048	2.584	7	5	---	1488
37.0	22.581	0.107	2.157	7	7	32	---
38.0	33.150	0.258	2.081	11	7	34	---
39.0	31.832	0.308	1.911	10	7	34	---
39.9	10.583	0.036	3.368	10	3	---	918
40.9	8.260	0.041	4.719	4	1	---	654
42.1	8.823	0.081	5.490	4	1	---	710
43.1	9.030	0.085	5.776	8	3	---	726
44.0	9.130	0.087	5.923	5	4	---	731
45.0	9.452	0.100	5.891	4	5	---	761
46.0	9.644	0.117	6.090	4	5	---	776
47.0	9.826	0.116	6.134	4	5	---	790
48.0	10.044	0.113	6.266	4	5	---	808
49.0	9.691	0.091	5.968	6	4	---	763
49.4	9.941	0.081	5.879	6	4	---	788

Appendix 9.2

OLSON ENGINEERING, Inc.

SPECIALIZING IN CONDITION EVALUATION OF THE CIVIL STRUCTURE & INFRASTRUCTURE



www.olsonengineering.com

November 14, 2005

Southern Earth Sciences, Inc.
11519 Cloverland Avenue
Baton Rouge, LA 70809

Attn: Mr. Scott H. Slaughter, P.E.
Tel: (225) 751-1727
Fax: (225) 752-1467
Cell: (225)252-6348
Email: sslaughter@soearth.com

Re: Consulting on Parallel Seismic/ Seismic Cone Penetrometer (PS/SCPT) Data
Evaluation of 4 PS/SCPT Soundings
17th St. and London St. Levees
New Orleans, LA
Olson Job No. 1896

Gentlemen:

As requested the following contains the results of our analyses of the subject PS/SCPT data. You will find attached the document containing all of the Parallel Seismic data sets with the interpretations on them. You will also find a table summarizing the results from the four sounding PS/SCPT data sets that we have recieved (CPT01, CPT03, CPT04, and LCPT01), with x and y both orientations. The table presents the results as well as the PS/SCPT physical locations in New Orleans, as per Scott Slaughter's directions. If there are any questions or comments concerning these results please feel free to contact either Larry D. Olson, Dennis Sack, or Hunter Yarbrough with Olson Engineering. Thank you for the opportunity to work on this project with you.

Respectfully Submitted,
Olson Engineering, Inc.

Larry D. Olson, P.E.
Principal Engineer

12401 W. 49th Ave., Wheat Ridge, CO 80033-1927 USA
PHONE: 303.423.1212 FAX: 303.423.6071

Southern Earth Sciences
Table I: Summary of Sheet Pile Tip Depths
From Parallel Seismic Testing of New Orleans Levees Using PS/CPT

FIGURE NAME	FIGURE NUMBER	SITE	ORIENTATION	DEPTH FROM GROUND LEVEL
CPT01	1	17th Street	Blue	13.7 ft
CPT01	2	17th Street	Red	14.1 ft
CPT03_Bl	3	17th Street	Blue	13.4 ft
CPT03_Re	4	17th Street	Red	14.2 ft
CPT04_Bl	5	17th Street	Blue	13.7 ft
CPT04_Re	6	17th Street	Red	13.9 ft
LCPT01BL	7	London	Blue	13.5 ft
LCPT01RE	8	London	Red	13.7 ft

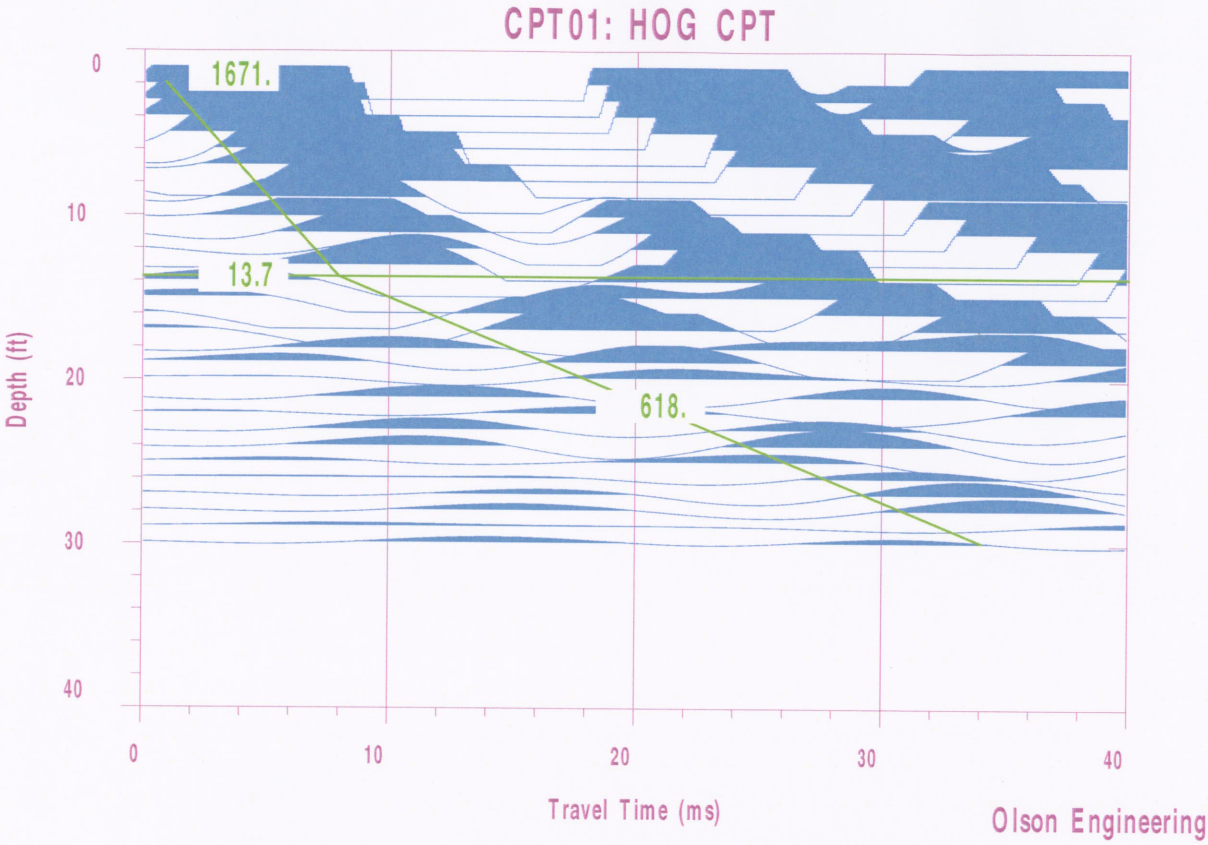


Figure 1: 17th Street, CPT01, Blue Orientation

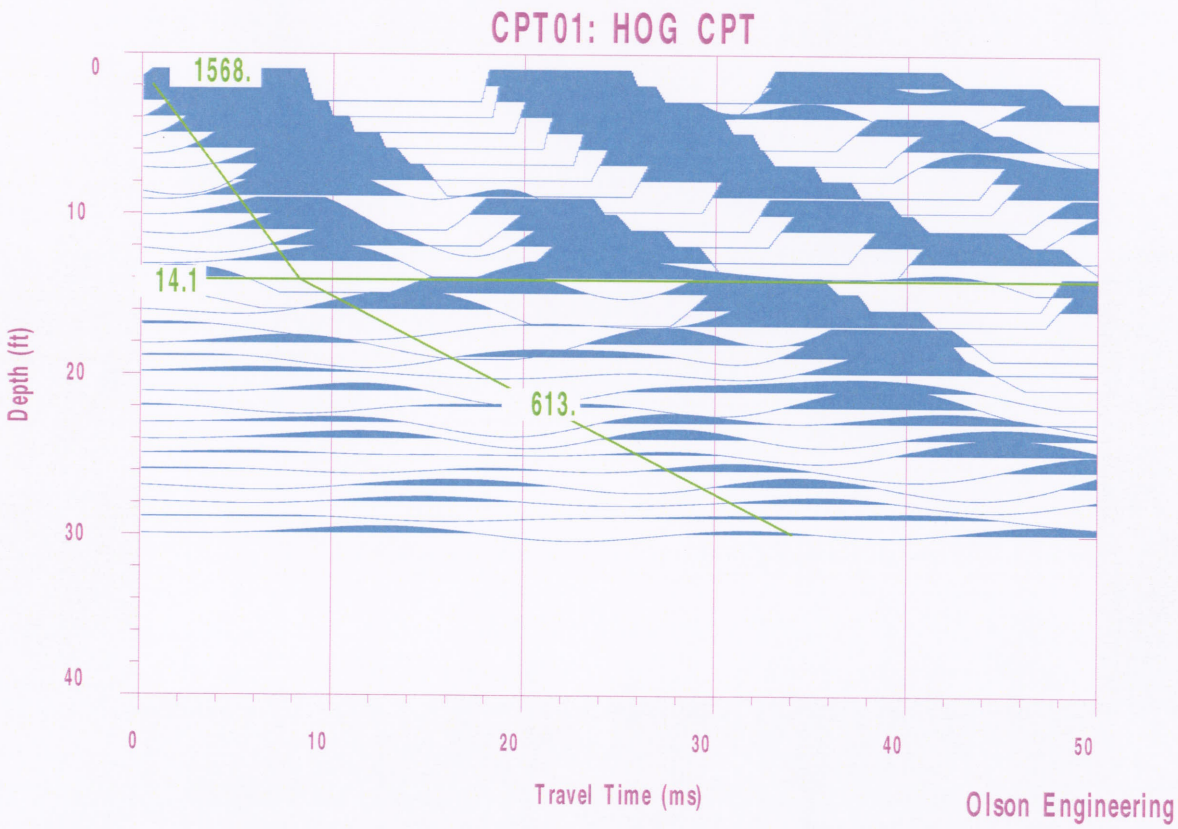


Figure 2: 17th Street, CPT01, Red Orientation

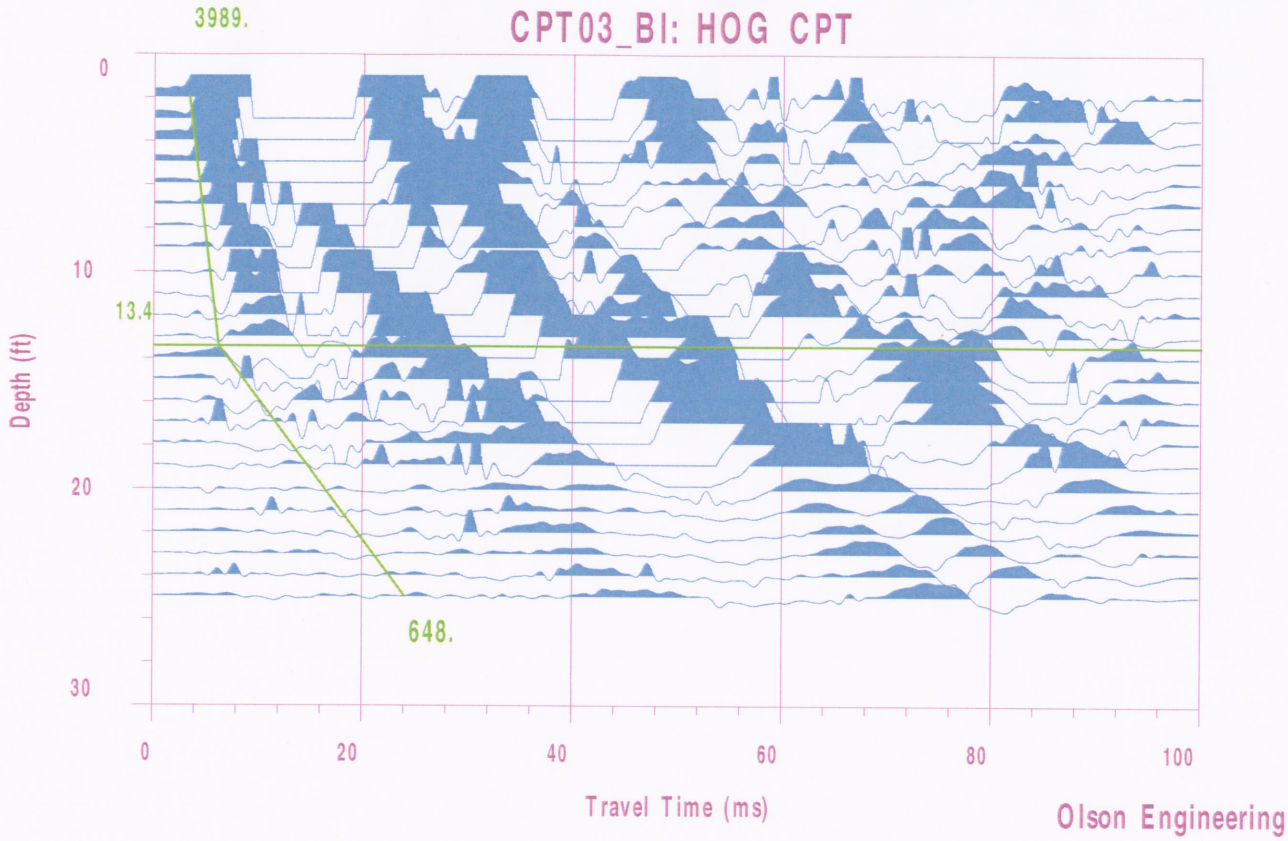


Figure 3: 17th Street, CPT03, Blue Orientation

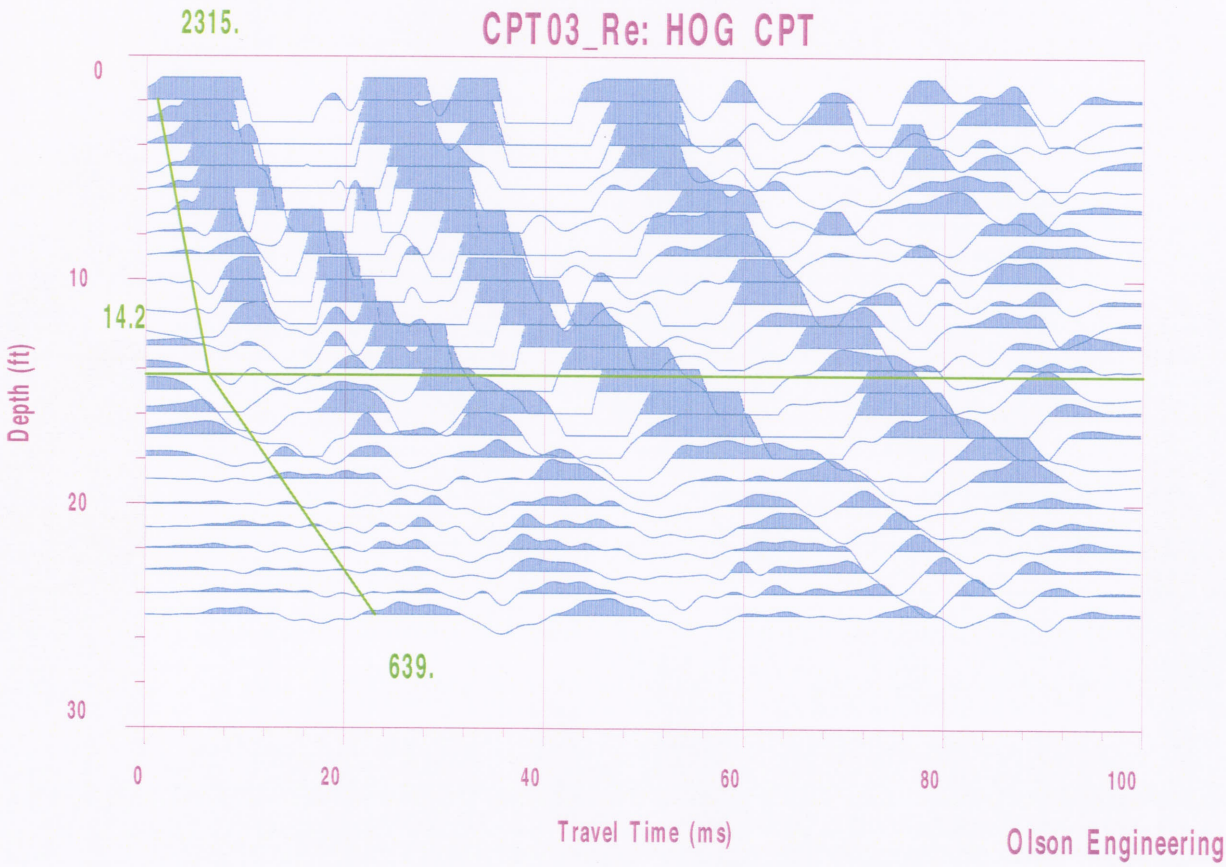


Figure 4: 17th Street, CPT03, Red Orientation

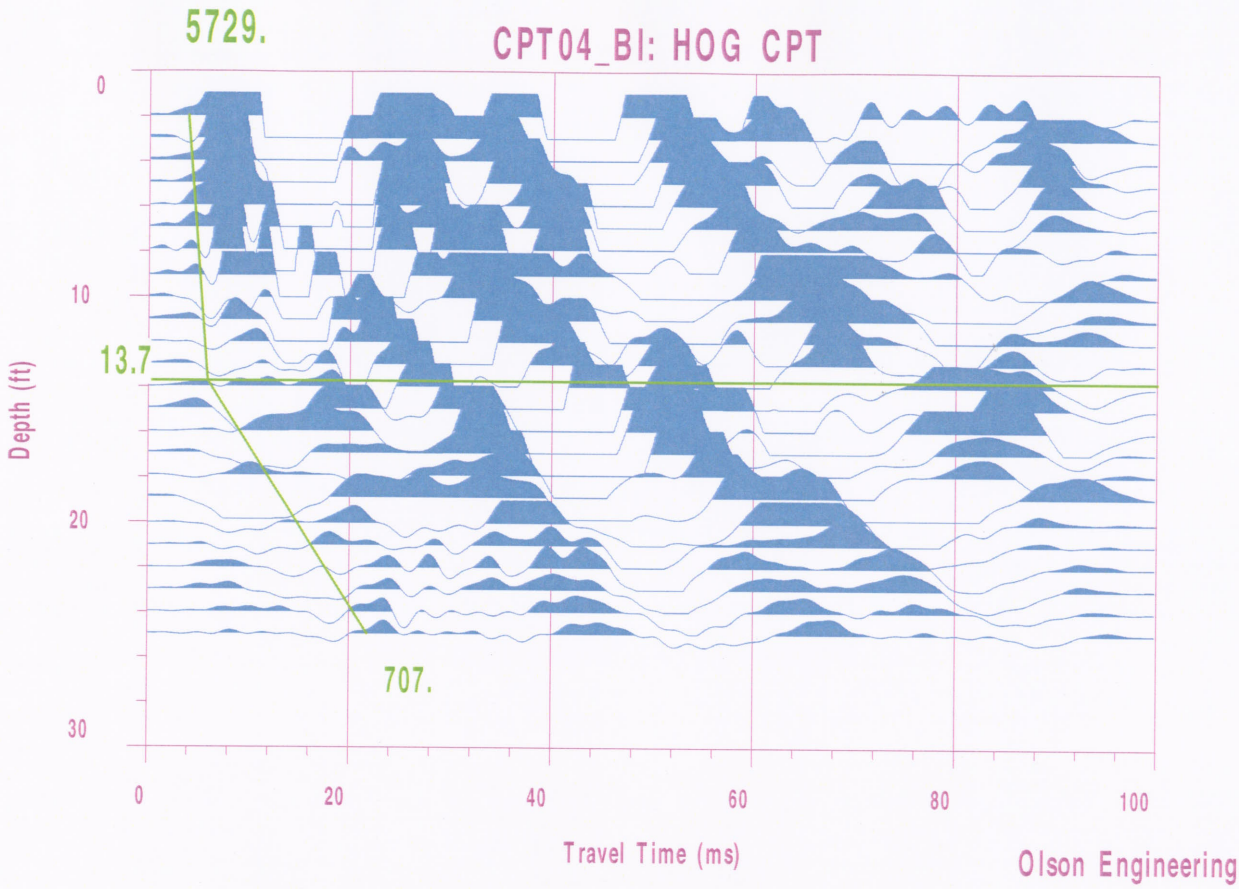


Figure 5: 17th Street, CPT04, Blue Orientation

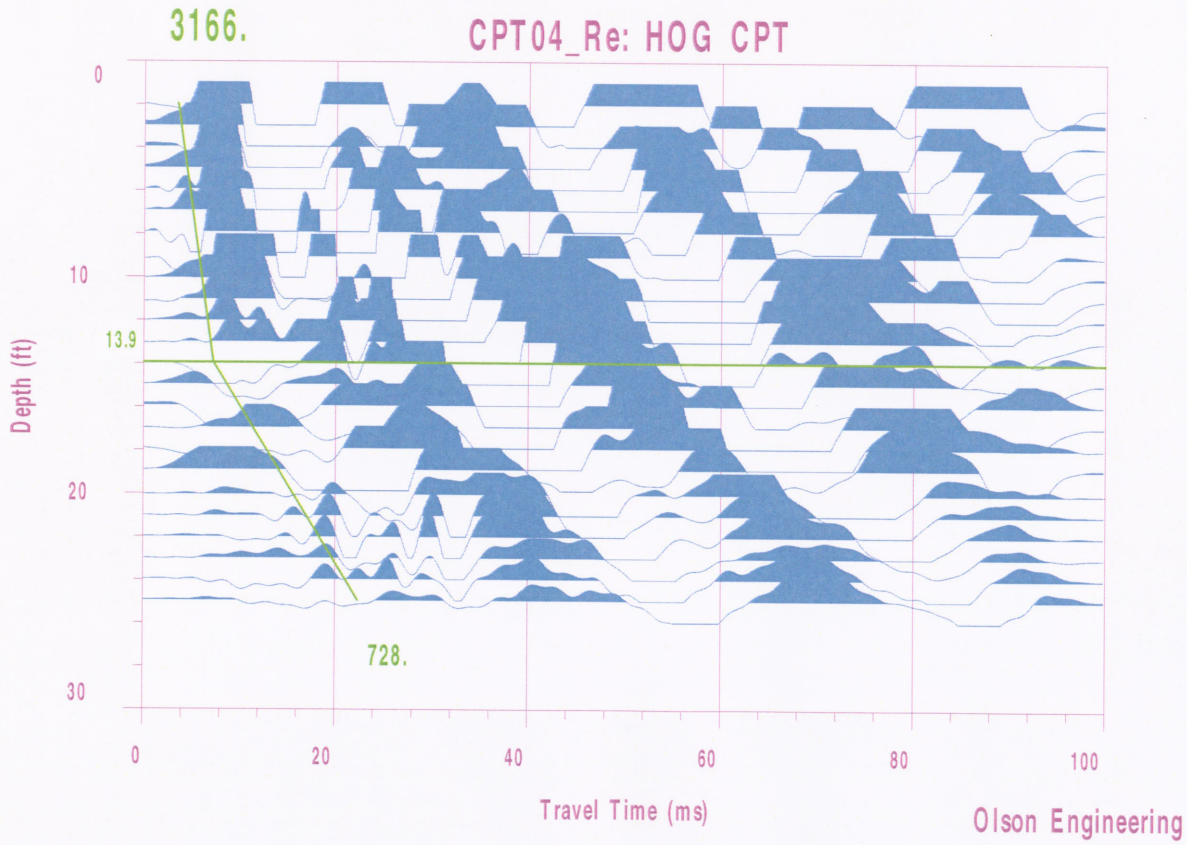


Figure 6: 17th Street, CPT04, Red Orientation

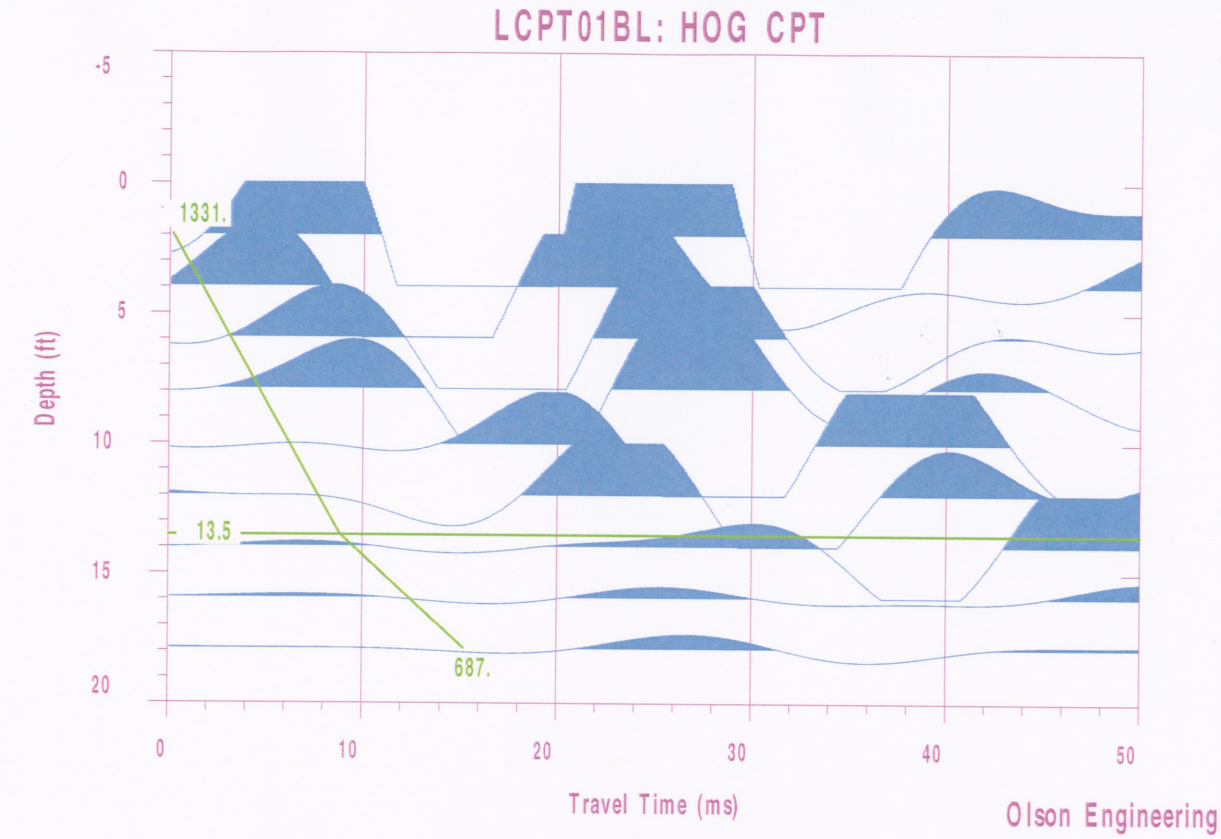


Figure 7: London Street, LCPT01, Blue Orientation

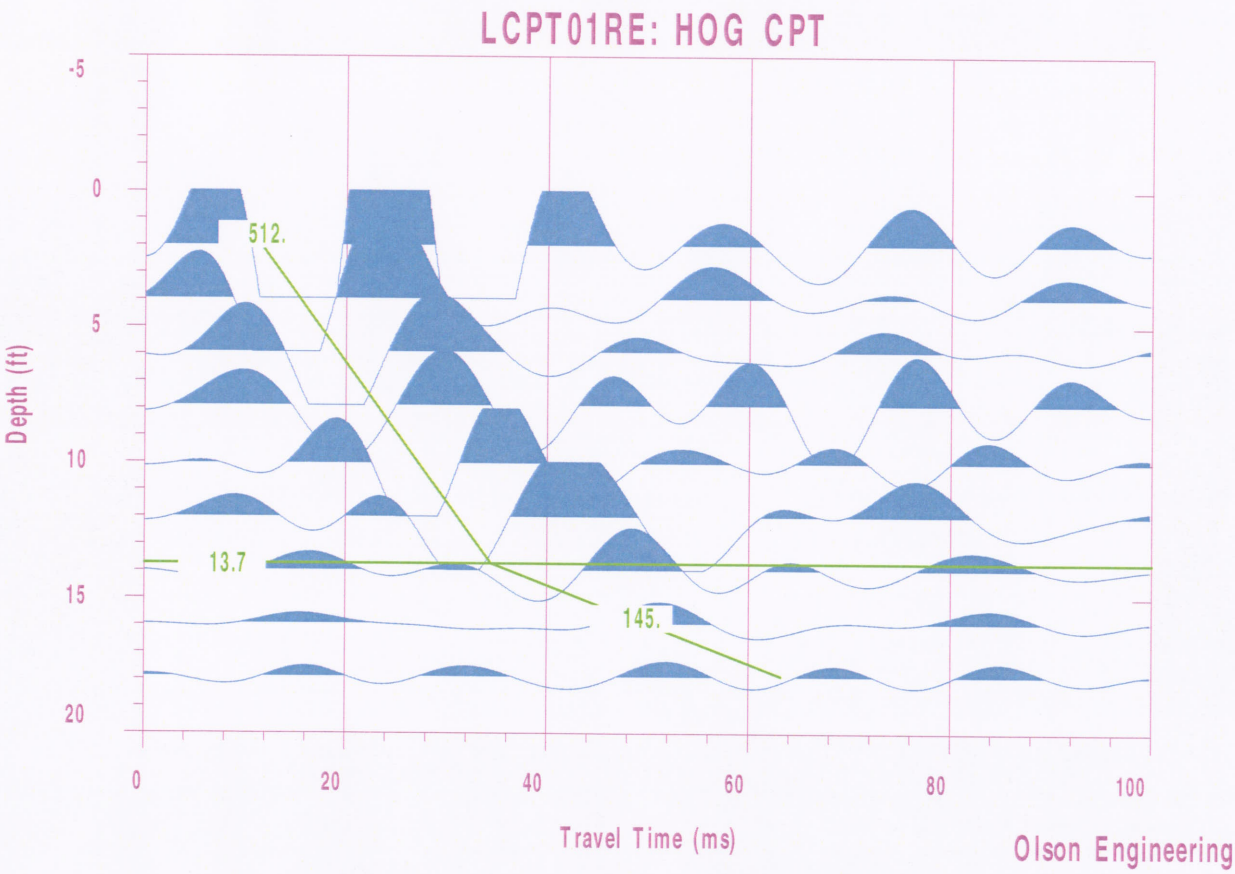


Figure 8: London Street, LCPT01, Red Orientation

Appendix 9.3

Lightweight CPT rig at 17th Street Canal Location

10-biaxial Seismic Cone and CPT Data Acquisition Unit

Undisturbed Sampling

Sample Extruder Assembly

SOUTHERN EARTH SCIENCES, inc.

Appendix 9.4

SOUTHERN EARTH SCIENCES, inc.

January 5, 2006

Louisiana State University
Center for the Study of Public Health Impacts of Hurricanes
Suite 3221 CEBA Building
Baton Rouge, Louisiana 70803

Attn: Ivor van Heerden, Ph.D.
Director

Re: Result of Non-Destructive Testing and Subsurface Investigation (Phase II)
Levee System
New Orleans, Louisiana
SESI Project No.: B05-157

Dear Mr. van Heerden:

Southern Earth Sciences, Inc. (SESI) has completed the Phase II work on the referenced project. The results of our non-destructive testing (NDT) program, laboratory analysis, and subsurface investigation are presented herein. The primary purpose of this Phase II work was to verify the testing procedures used in our original investigation and presented in our report dated November 22, 2005. Authorization to proceed for this phase of the project was authorized by you.

PROJECT INFORMATION

Since issuing our November 22, 2005 report, the Corps of Engineers has extracted sheet piling from the 17th Street Canal Levee breach area to verify the embedded depths. We understand that the sheet piles at this location were measured to be approximately 23.5 ft in overall length with a final tip elevation at -17 ft. Though the sheet piles were not extracted from our actual previous test locations, the measured sheet pile depths at the breach area conflict with the results previously presented in our report of having a final sheet pile tip elevation at approximately -10 ft.

Prior to our original work, we understand the Corps of Engineers contracted Olson Engineering, Inc. to perform an independent NDT investigation to evaluate the embedded sheet pile lengths using Parallel Seismic (PS) methods. We understand the initial results of the COE investigation also indicated the sheet piles to have a final tip elevation at approximately -10 ft. Olson Engineering analyzed both the COE data and our prior PS/CPT data collected during our previous investigation for the embedded sheet pile depths.

Due to the discrepancy between the measured sheet pile depths and those previously reported by Olson, we decided to revisit the site and perform additional testing using both PS and PS/CPT methods. Mr. Larry Olson accompanied our field crews to the site. In order to verify the NDT test results, tests were performed at the IHNC Levee and 17th Street Canal Levee where segments of the sheet piles were either exposed and could be directly measured, or had been purposely extracted for the same purpose. This additional investigation was conducted in order to verify the NDT results and to assess if modifications were necessary in the testing procedure or data analysis methods.

Our field activities during this phase of the project were performed under the direction of Mr. Billy Prochaska, P.E. during the period of December 21-22, 2005.

FIELD TESTING PROCEDURES

Both Parallel Seismic (PS) and Parallel Seismic/Cone Penetration Test (PS/CPT) methods were used during this phase of our investigation. In the PS test procedure a hydrophone is lowered into a cased borehole to measure wave energy transmitted from the foundation element when it is struck with an instrumented hammer. Mr. Larry Olson collected the PS data and we understand these test results will be presented in a separate report prepared independently by Olson Engineering, Inc.

SESI used the PS/CPT method to evaluate the embedded lengths of the steel sheet piles. The PS/CPT method is similar to the PS method except that a bi-axial geophone seismic piezocone is advanced into the soil adjacent to the foundation element. The element is impacted with an instrumented hammer and the wave energy is measured by the seismic piezocone and recorded on a computer. The resulting plot of wave energy vs. depth is then analyzed to determine the element depth.

A Geoprobe 6625 electronic CPT rig was used to perform the non-destructive testing and subsurface investigation. This lightweight rig weighs about 4.5-tons yet has a full 20-tons pushing capacity. Automated anchors are augered into the soil to provide the necessary reaction for advancing the cone.

Inherit with the PS/CPT test method is the ability to determine subsurface conditions using the seismic piezocone. An instrumented cone attached to a series of rods is continuously pushed into the soil at a constant rate. Built-in load cells are used to continuously measure pore pressure, tip resistance and unit skin friction during penetration where the data is then recorded on a computer.

SUBSURFACE INVESTIGATION

Cone Penetration Tests (CPT)

A total of four (4) CPT soundings were performed during this current investigation to assess subsurface conditions. A total of seven (7) CPT soundings were performed during our initial investigation. The CPT soundings were conducted simultaneously with the PS/CPT test process. All soundings were performed in general accordance with ASTM Specification D-5778. The station numbering convention marked in the field was used to locate the soundings performed during our current investigation. GPS coordinates were recorded at the previous test locations using a hand-held Garmin GPS 72.

The soundings conducted as part of this current investigation were performed at the approximate locations shown below:

Current Investigation (Dec. 21-22, 2005)

TEST NUMBER	LOCATION
CPT-01A 17th Street Canal Levee	Sta. 20+78 South end of breach
CPT-02A* 17th Street Canal Levee	Sta. 20+77 South end of breach
CPT-03A 17th Street Canal Levee	Sta. 4+08 Approx. 200 ft south of breach
CPT-01B IHNC	Sta. 17+11

*was performed about 1-ft from CPT-01A. Subsurface data showed signs of possible disturbance therefore CPT data for this sounding is not included in this report.

Previous Investigation (Nov. 9-11, 2005)

TEST NUMBER	LOCATION
CPT-01 17th Street Canal Levee	N 30° 00.890ô W 90° 07.293ô
CPT-02 17th Street Canal Levee	N 30° 00.922ô W 90° 07.289ô
CPT-03 17th Street Canal Levee	N 30° 00.945ô W 90° 07.289ô
CPT-04 17th Street Canal Levee	N 30° 00.967ô W 90° 07.286ô
CPT-05 17th Street Canal Levee	N 30° 00.970ô W 90° 07.244ô
LCPT-01 London Avenue Levee @ Maribeau	N 30° 00.538ô W 90° 04.167ô
LCPT-02 London Avenue Levee @ R. E. Lee	N 30° 01.236ô W 90° 04.267ô

The CPT Log sheets are attached in the Appendix, graphically showing the cone tip resistance, side friction, friction ratio, equivalent N-value and interpreted soil behavior type at each sounding location. The soil behavior type was determined using the methods of Robertson & Campanella (1983).

Also presented in the Appendix are tabulated data sheets for the CPT soundings showing additional soil parameters such as internal friction angle and undrained shear strength. These parameters were derived using a software program developed for SESI and using generally accepted interpretation methods as shown on the tabulated data cover sheets. In deriving the tabulated soil parameters, a CPT data averaging technique was used over 1-foot intervals.

The tabulated data is presented for informational purposes only and users should exercise sound judgment when using this information for the interpretation of reliable soil parameters. We highly recommend the engineer be thoroughly familiar with and fully understand the capabilities of the CPT test before using this information for design purposes.

Soil Test Boring

During our previous investigation, one (1) soil test boring was performed at the B-3 (CPT-03) location to assess subsurface conditions for verification purposes. Samples were obtained at 2-foot intervals to a depth of ten feet using conventional auger drilling methods with our lightweight CPT rig.

Undisturbed samples were recovered from the various cohesive materials with a 3-inch diameter thin walled Shelby tube (ASTM D-1587). The samples were extruded in the field where they were visually classified by the field technician. Representative portions of the samples were then wrapped and sealed to preserve their natural characteristics during transportation to the laboratory for physical testing.

Laboratory tests were performed on the collected samples to determine their physical characteristics. The laboratory test results have been tabulated below:

FIELD TESTING PROCEDURES

Both Parallel Seismic (PS) and Parallel Seismic/Cone Penetration Test (PS/CPT) methods were used during this phase of our investigation. In the PS test procedure a hydrophone is lowered into a cased borehole to measure wave energy transmitted from the foundation element when it is struck with an instrumented hammer. Mr. Larry Olson collected the PS data and we understand these test results will be presented in a separate report prepared independently by Olson Engineering, Inc.

SESI used the PS/CPT method to evaluate the embedded lengths of the steel sheet piles. The PS/CPT method is similar to the PS method except that a bi-axial geophone seismic piezocone is advanced into the soil adjacent to the foundation element. The element is impacted with an instrumented hammer and the wave energy is measured by the seismic piezocone and recorded on a computer. The resulting plot of wave energy vs. depth is then analyzed to determine the element depth.

A Geoprobe 6625 electronic CPT rig was used to perform the non-destructive testing and subsurface investigation. This lightweight rig weighs about 4.5-tons yet has a full 20-tons pushing capacity. Automated anchors are augered into the soil to provide the necessary reaction for advancing the cone.

Inherit with the PS/CPT test method is the ability to determine subsurface conditions using the seismic piezocone. An instrumented cone attached to a series of rods is continuously pushed into the soil at a constant rate. Built-in load cells are used to continuously measure pore pressure, tip resistance and unit skin friction during penetration where the data is then recorded on a computer.

SUBSURFACE INVESTIGATION

Cone Penetration Tests (CPT)

A total of four (4) CPT soundings were performed during this current investigation to assess subsurface conditions. A total of seven (7) CPT soundings were performed during our initial investigation. The CPT soundings were conducted simultaneously with the PS/CPT test process. All soundings were performed in general accordance with ASTM Specification D-5778. The station numbering convention marked in the field was used to locate the soundings performed during our current investigation. GPS coordinates were recorded at the previous test locations using a hand-held Garmin GPS 72.

The soundings conducted as part of this current investigation were performed at the approximate locations shown below:

Current Investigation (Dec. 21-22, 2005)

TEST NUMBER	LOCATION
CPT-01A 17th Street Canal Levee	Sta. 20+78 South end of breach
CPT-02A* 17th Street Canal Levee	Sta. 20+77 South end of breach
CPT-03A 17th Street Canal Levee	Sta. 4+08 Approx. 200 ft south of breach
CPT-01B IHNC	Sta. 17+11

*was performed about 1-ft from CPT-01A. Subsurface data showed signs of possible disturbance therefore CPT data for this sounding is not included in this report.

Previous Investigation (Nov. 9-11, 2005)

TEST NUMBER	LOCATION	CALCULATED SHEET PILE EMBEDMENT*
CPT-01 17th Street Canal Levee	N 30° 00.890ô W 90° 07.293ô	20 ft
CPT-03 17th Street Canal Levee	N 30° 00.945ô W 90° 07.289ô	20 ft
CPT-04 17th Street Canal Levee	N 30° 00.967ô W 90° 07.286ô	20.5 ft
LCPT-01 London Avenue Levee @ Maribeau	N 30° 00.538ô W 90° 04.167ô	15 ft

*embedded lengths are measured from ground surface at each test location.

DISCUSSION OF ANALYSIS RESULTS

Overall, our analysis of the PS/CPT data collected at the various sites yielded results that we consider are rather definitive and are in general agreement with measured foundation depths. Specific comments regarding the analysis results at a few of the test locations are outlined below.

CPT-01B - IHNC: We consider the PS/CPT data for this test location to be of fair to poor quality. Significant filtering and amplifying of the wave energy was necessary in order for us to perform our analysis for the sheet pile depth. We understand the sheet pile wall at this test location had experienced translational movements of up to about 7-inches as a result of Hurricane Katrina. This condition may have caused the subsurface soils not to be in direct contact with the entire sheet pile face and have contributed to the reduced data quality. Data collected from the two additional tests performed at this site yielded analysis results that we do not consider to be definitive and are not included in this report.

LCPT-01 – London Avenue @ Maribeau: Due to time constraints during our previous investigation, the PS/CPT data for this test location was collected at two feet intervals rather than the typical 1 foot interval. Additionally, the data was collected only to a depth 18 feet. While the analysis of the data indicates a sheet pile embedment that we consider rather apparent and definitive, we caution that the wider testing interval used and relatively shallow testing depth can significantly affect the overall accuracy of the analysis results.

GENERAL COMMENTS

Professional judgments are presented in this report. These are based partly on our evaluations of field information gathered, partly on our understanding of the characteristics of the project, and partly on our capabilities and experience using CPT and non-destructive testing methods.

Limitations exist for the capabilities of the NDT process used on this project and we do not guarantee the test results in any respect. The NDT results should only be considered an investigative tool for evaluating the conditions of interest and should not be used in place of sound engineering judgment. Accordingly, any overall conclusions and project design decisions should not be based solely on the NDT results presented herein.

We appreciate this opportunity to be of service. If we can answer any questions, please call.

Very truly yours,

SOUTHERN EARTH SCIENCES, INC.

Scott H. Slaughter, P.E.
Branch Manager

cc: Randy Sharma
Billy Prochaska
Ed Preau (2 copies)

SHS/slj

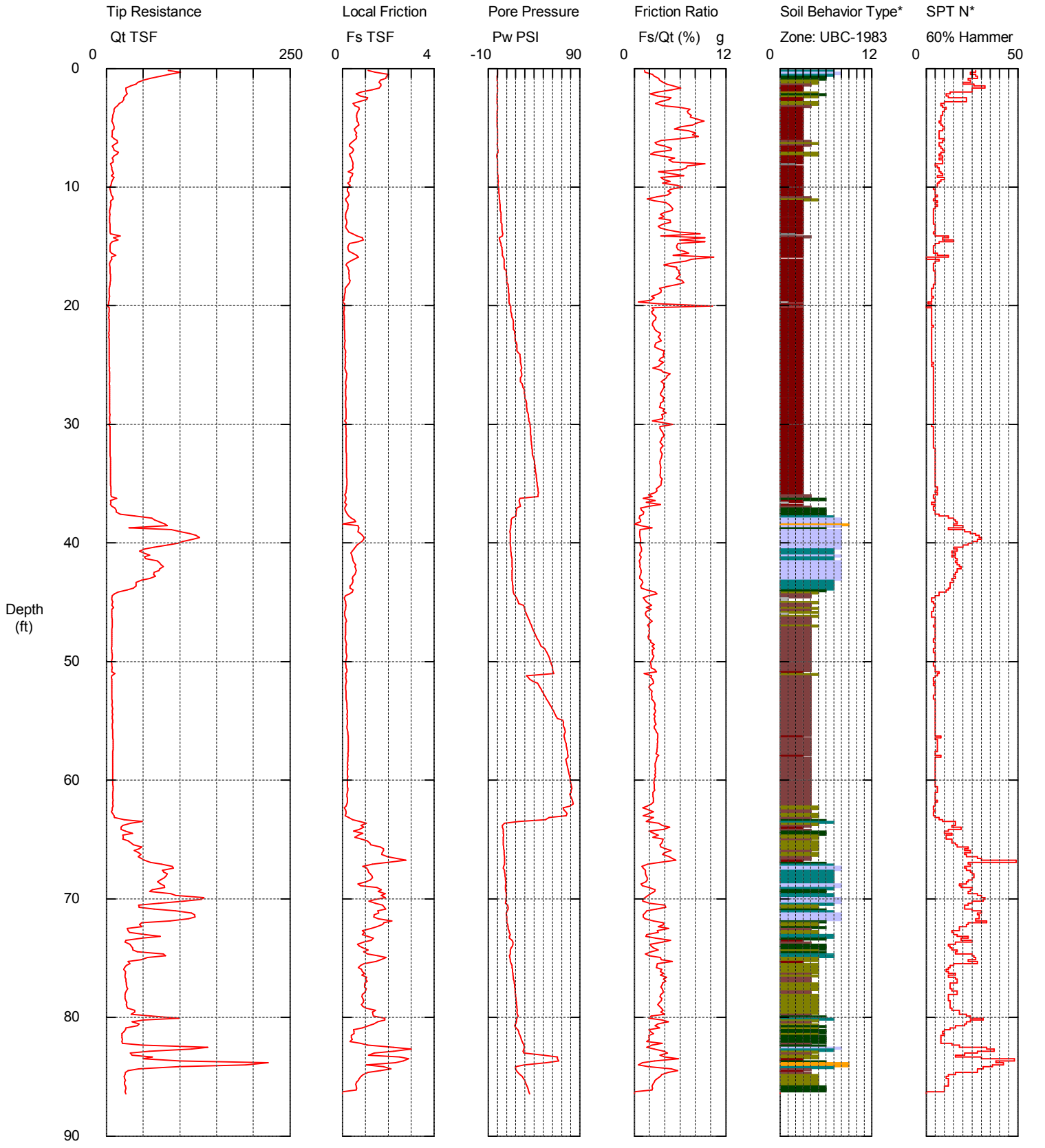
Appendix 1 CPT Log Sheets
Appendix 2 Tabulated Soil Parameters
Appendix 3 PS/CPT Data Analysis Results

Phase II CPT 01

Southern Earth Sciences, Inc.

Operator: WRP
Sounding: CPT-01
Cone Used: DDG0899

CPT Date/Time: 11/9/2005 2:56:22 PM
Location: 17th Street Canal Levee
Job Number: B05-157



Maximum Depth = 86.45 feet

Depth Increment = 0.164 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

A.IX

GPS Coordinates
N30 00.890' W90 07.293'

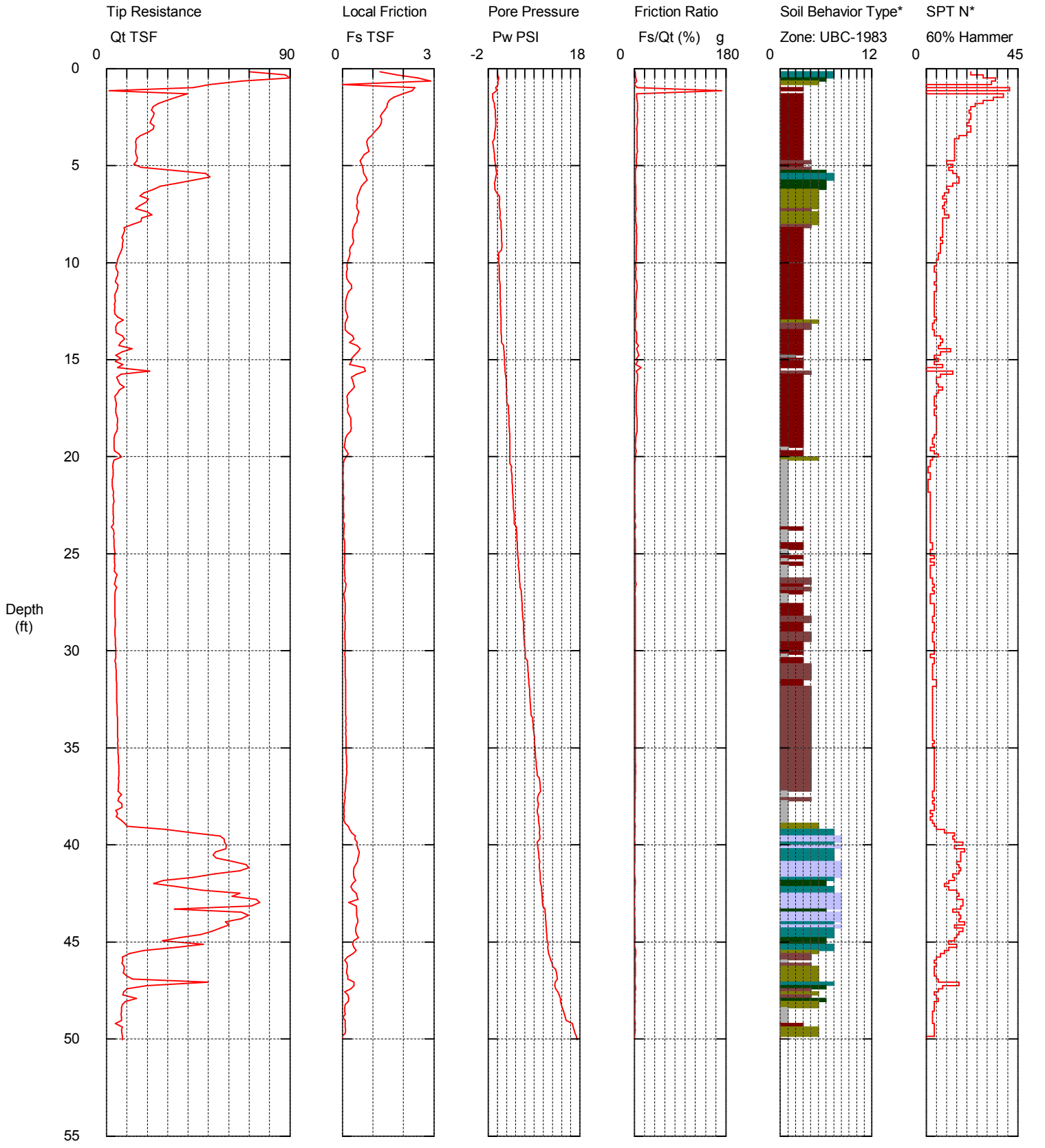
*Soil behavior type and SPT based on data from UBC-1983

Phase II CPT 02

Southern Earth Sciences, Inc.

Operator: WRP
Sounding: CPT-02
Cone Used: DDG0899

CPT Date/Time: 11/10/2005 10:44:18 AM
Location: 17th Street Canal Levee
Job Number: B05-157



Maximum Depth = 50.03 feet

Depth Increment = 0.164 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

A.IX

GPS Coordinates
N30 00.922' W90 07.289'

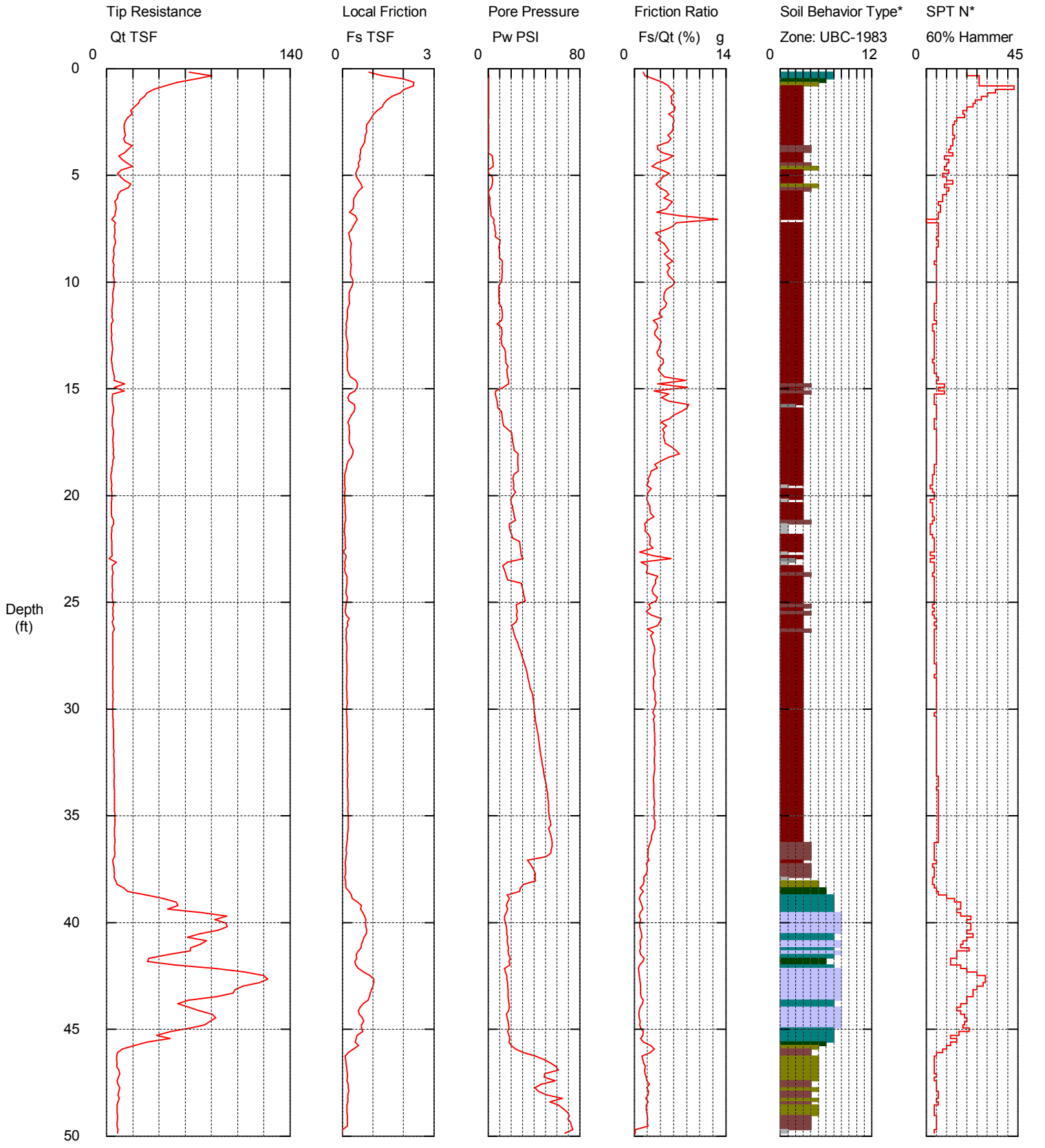
*Soil behavior type and SPT based on data from UBC-1983

Phase II CPT 03

Southern Earth Sciences, Inc.

Operator: WRP
Sounding: CPT-03
Cone Used: DDG0899

CPT Date/Time: 11/10/2005 12:30:00 PM
Location: 17th Street Canal Levee
Job Number: B05-157



Maximum Depth = 49.87 feet

Depth Increment = 0.164 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

A.IX

GPS Coordinates
N30 00.945' W90 07.289'

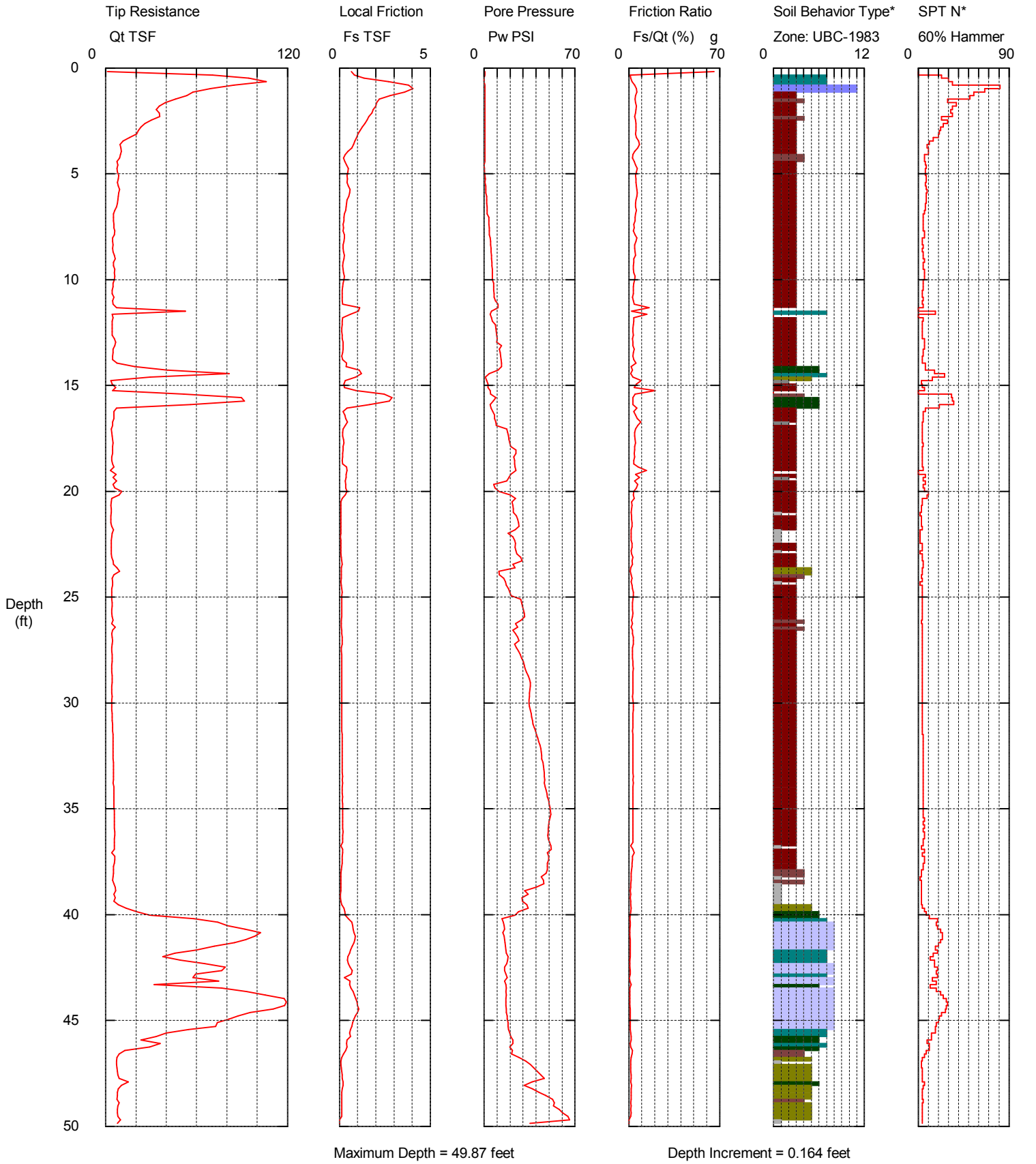
*Soil behavior type and SPT based on data from UBC-1983

Phase II CPT 04

Southern Earth Sciences, Inc.

Operator: WRP
Sounding: CPT-04
Cone Used: DDG0899

CPT Date/Time: 11/10/2005 3:19:01 PM
Location: 17th Street Canal Levee
Job Number: B05-157



Maximum Depth = 49.87 feet

Depth Increment = 0.164 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

A.IX

GPS Coordinates
N30 00.967' W90 07.286'

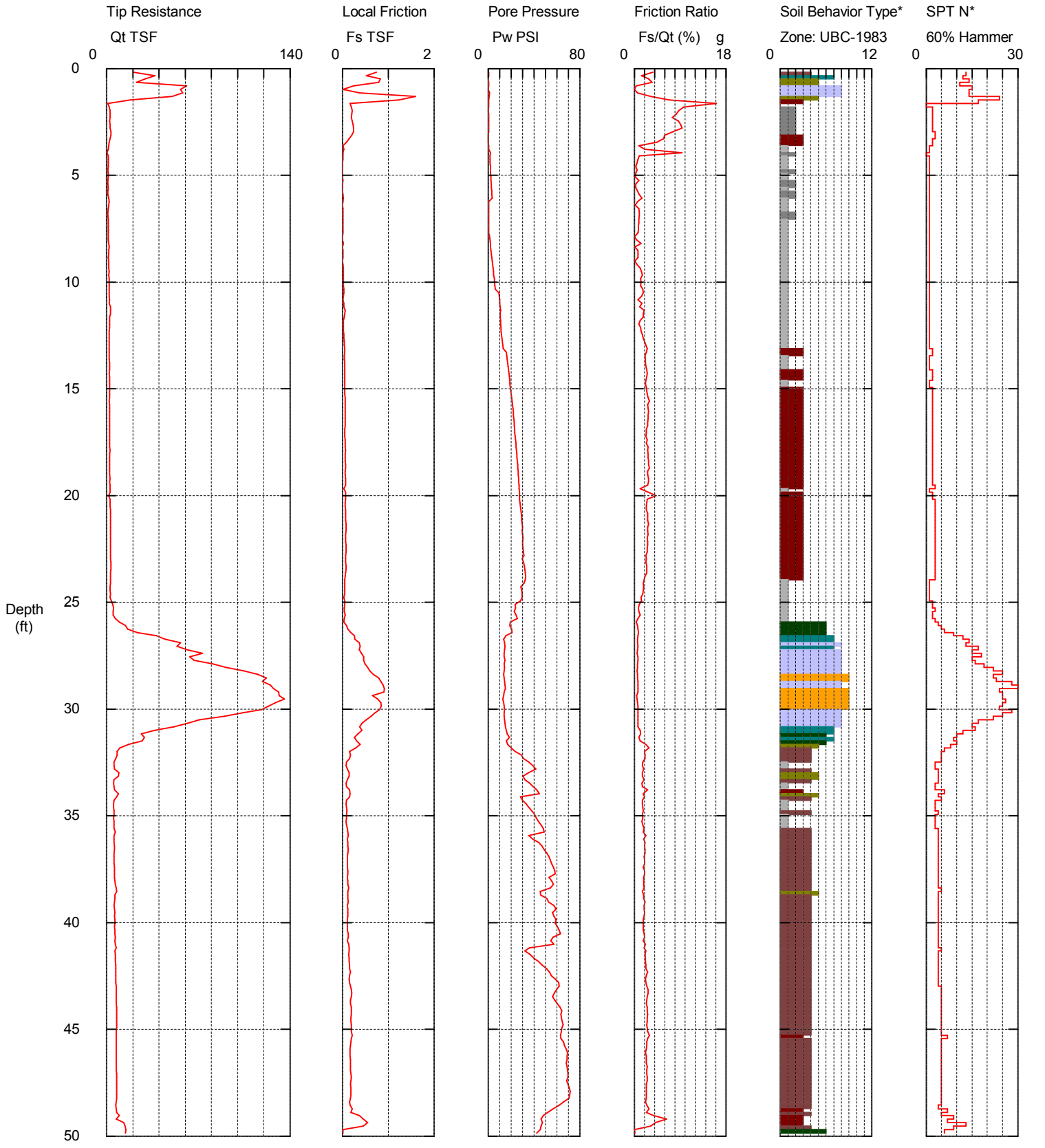
*Soil behavior type and SPT based on data from UBC-1983

Phase II CPT 05

Southern Earth Sciences, Inc.

Operator: WRP
Sounding: CPT-05
Cone Used: DDG0899

CPT Date/Time: 11/11/2005 11:35:46 AM
Location: 17th Street Canal Levee
Job Number: B05-157



Maximum Depth = 49.87 feet

Depth Increment = 0.164 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

A.IX

GPS Coordinates
N30 00.970' W90 07.244'

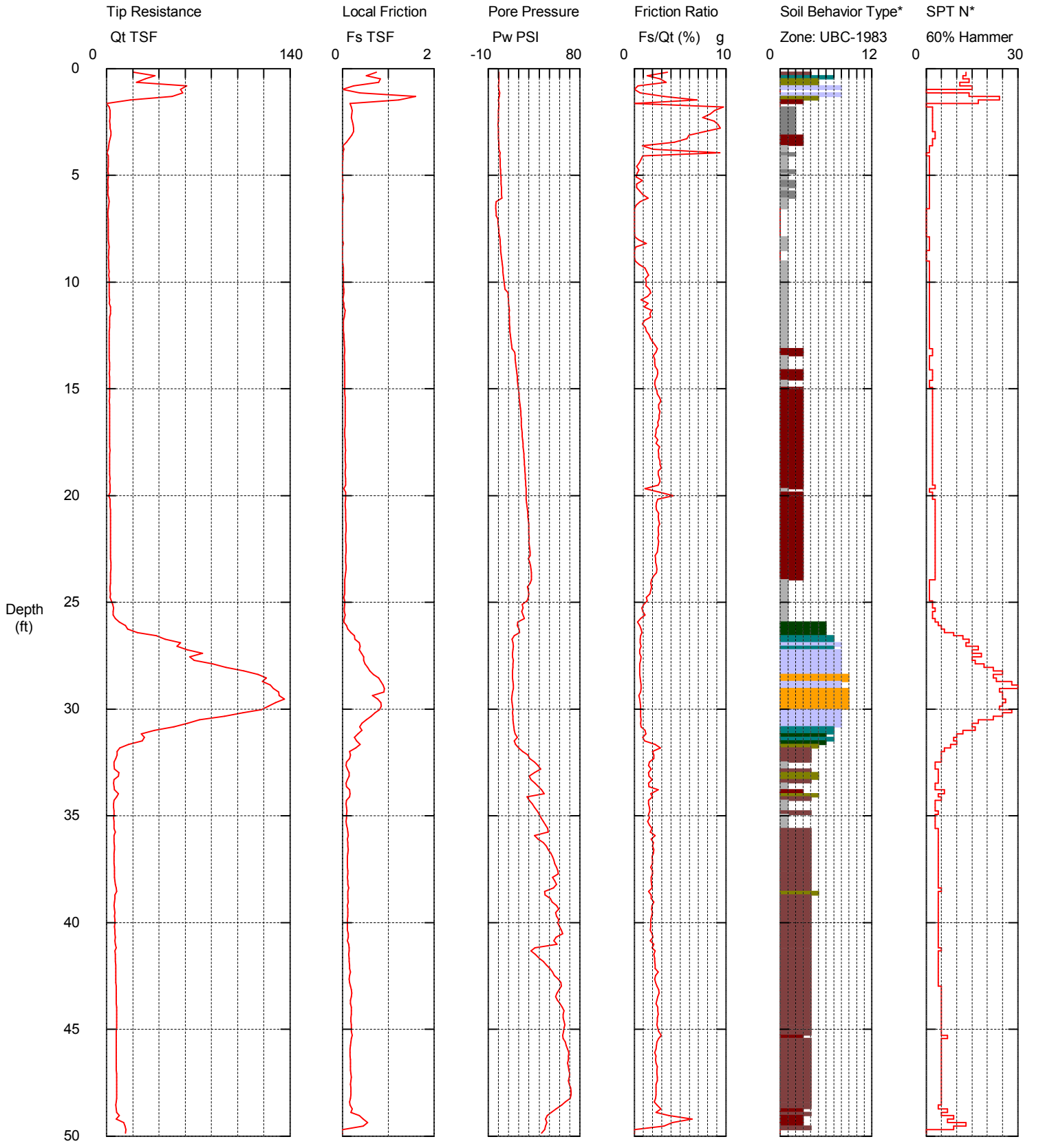
*Soil behavior type and SPT based on data from UBC-1983

Phase II CPT 05A

Southern Earth Sciences, Inc.

Operator: WRP
Sounding: CPT-05A
Cone Used: DDG0899

CPT Date/Time: 11/11/2005 11:35:46 AM
Location: 17TH STREET
Job Number: B05-157



Maximum Depth = 49.87 feet

Depth Increment = 0.164 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

A.IX

N 30° 00.970'
W 90° 07.244'

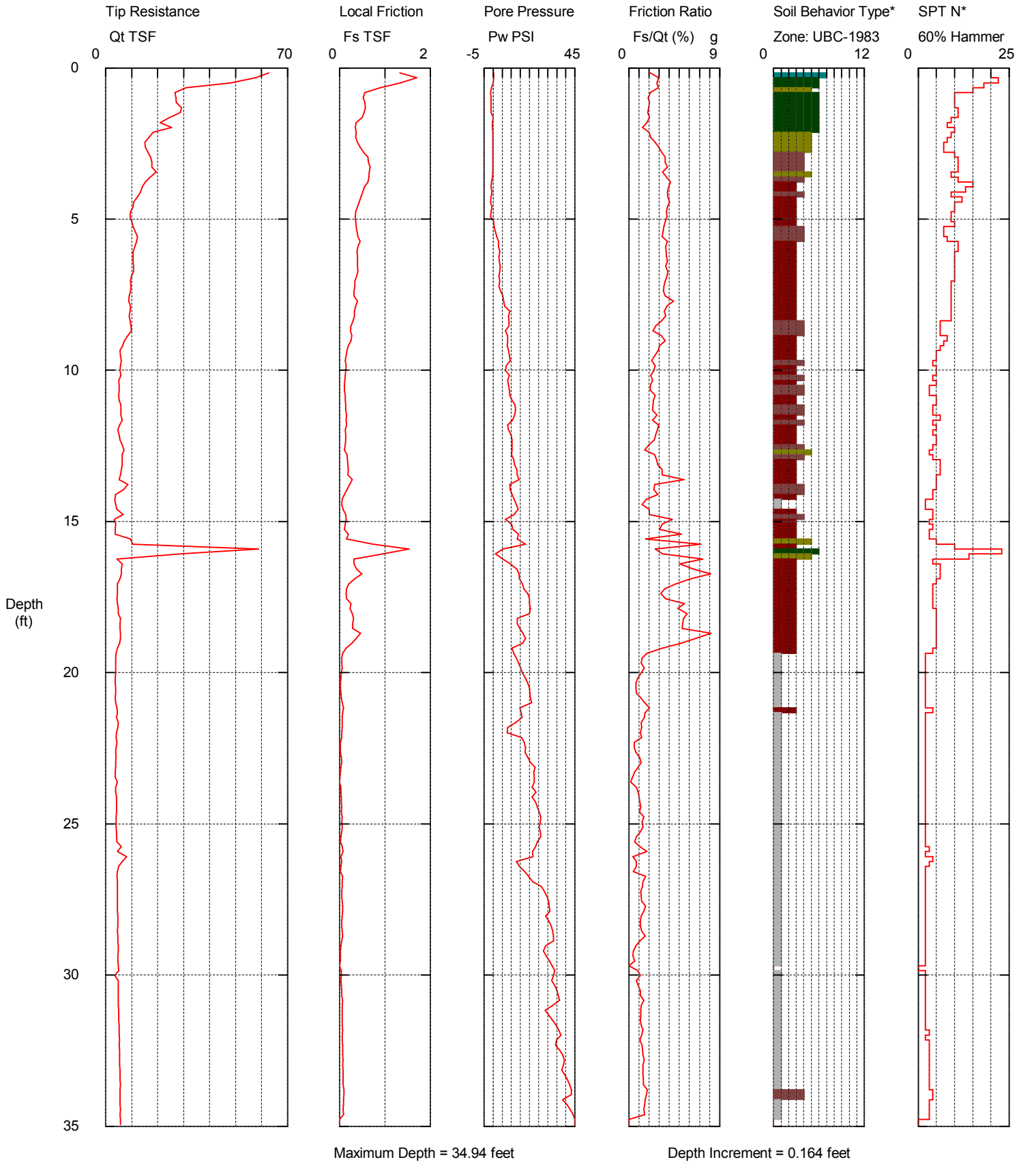
*Soil behavior type and SPT based on data from UBC-1983

Phase II CPT_1A

Southern Earth Sciences, Inc.

Operator: WRP
Sounding: CPT-1A
Cone Used: DDG0899

CPT Date/Time: 12/22/2005 9:50:17 AM
Location: 17th Street Canal Levee
Job Number: B05-157



Maximum Depth = 34.94 feet

Depth Increment = 0.164 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

A.IX

Sta. 20+78
South End of Breach

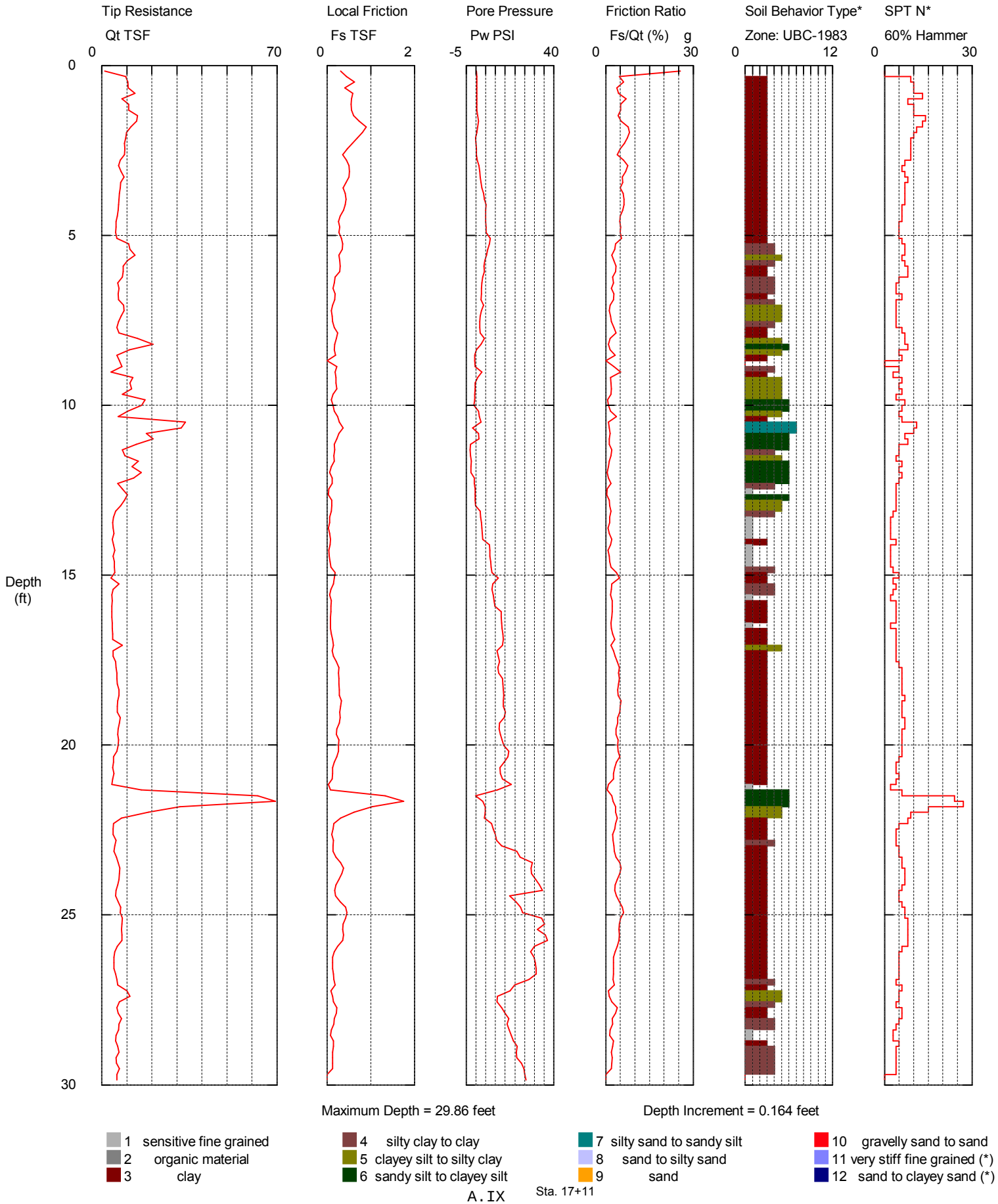
*Soil behavior type and SPT based on data from UBC-1983

Phase II CPT_1B

Southern Earth Sciences, Inc.

Operator: WRP
Sounding: CPT-1B
Cone Used: DDG0899

CPT Date/Time: 12/21/2005 11:21:37 AM
Location: IHNC
Job Number: B05-157



Maximum Depth = 29.86 feet

Depth Increment = 0.164 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

A.IX

Sta. 17+11

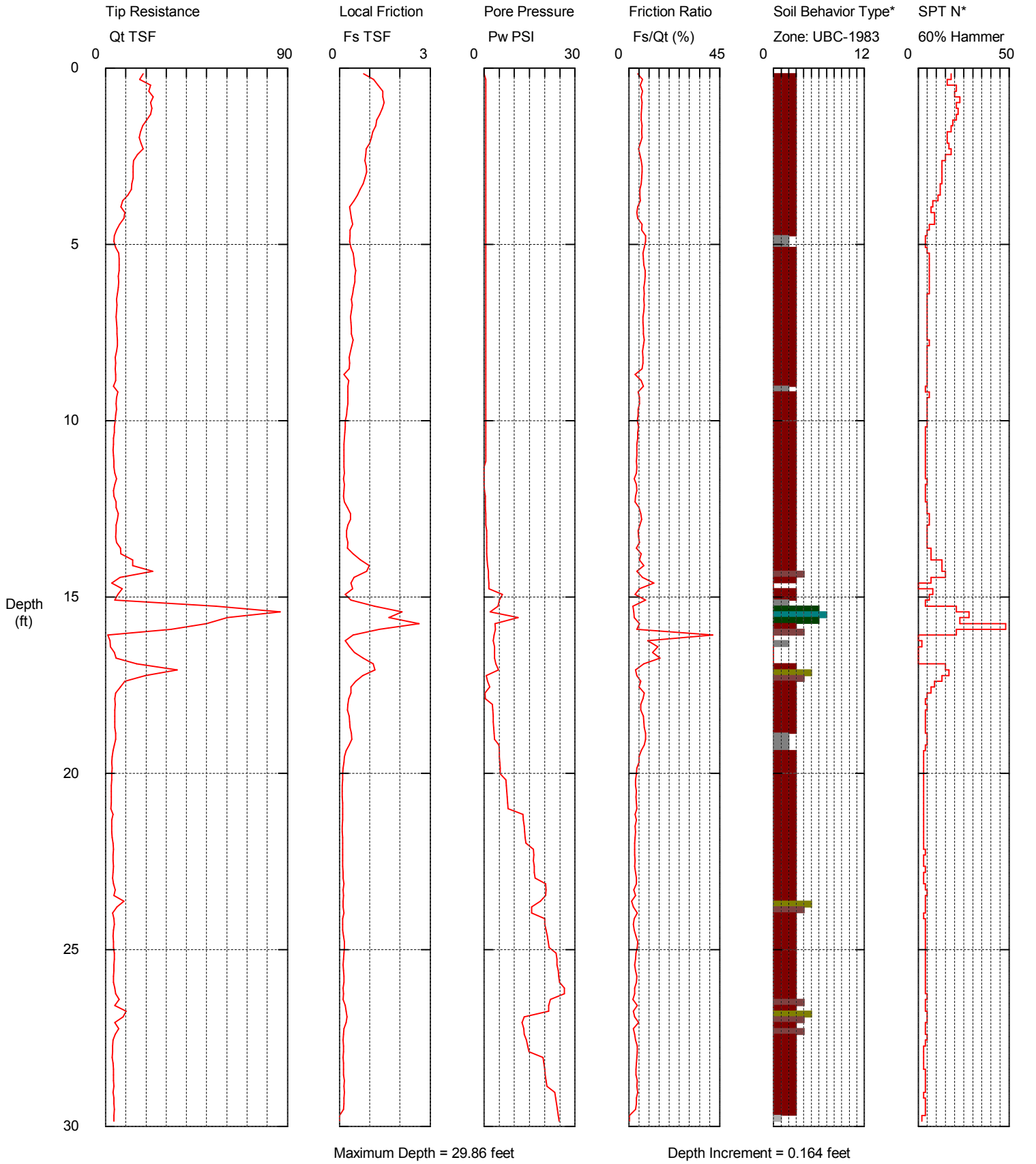
*Soil behavior type and SPT based on data from UBC-1983

Phase II CPT_3A

Southern Earth Sciences, Inc.

Operator: WRP
Sounding: CPT-3A
Cone Used: DDG0892

CPT Date/Time: 12/22/2005 2:34:15 PM
Location: 17th Street Canal
Job Number: B05-157



Maximum Depth = 29.86 feet

Depth Increment = 0.164 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

A.IX

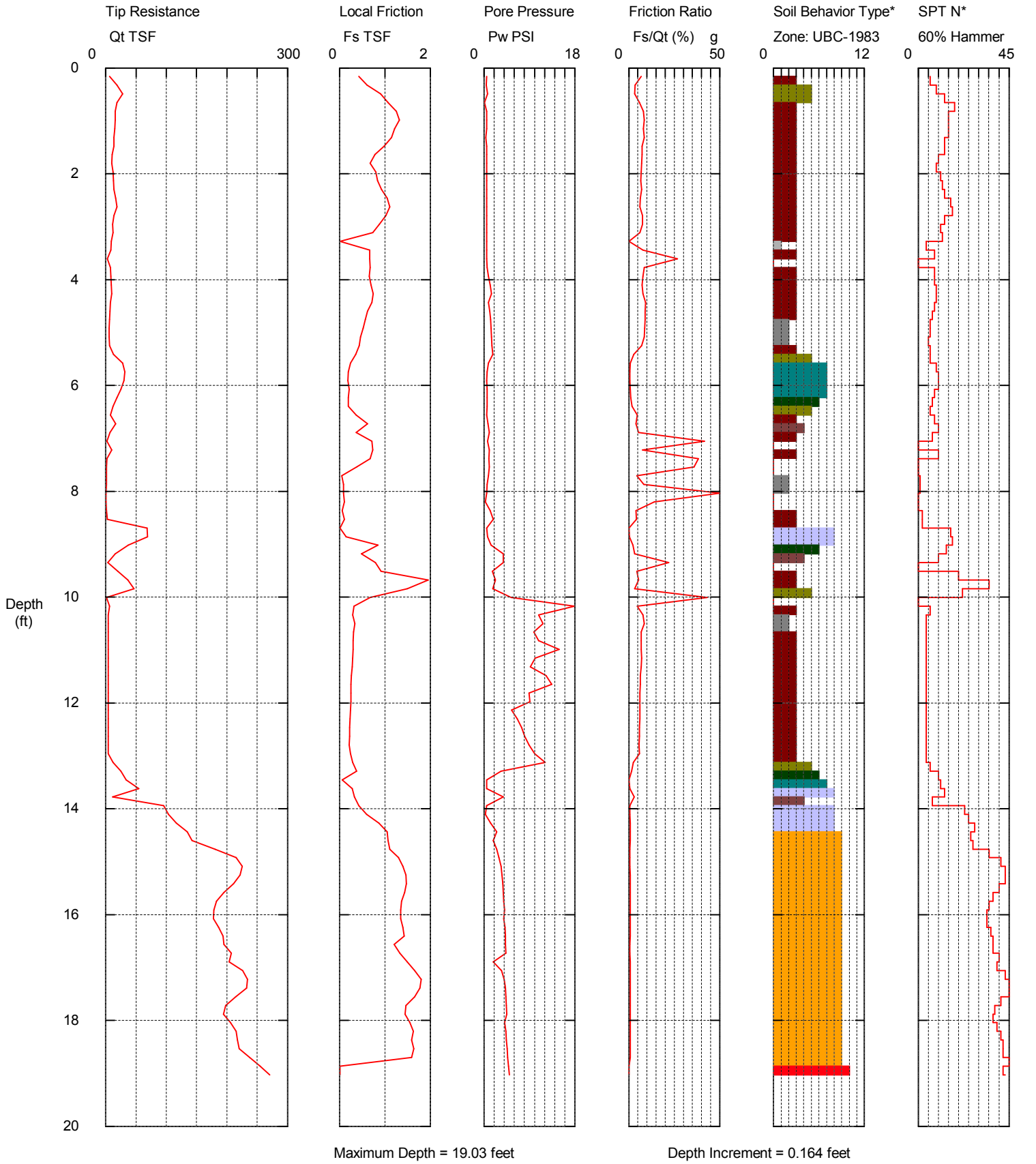
Sta. 4+08
Approx. 200 ft. south of breach

Phase II LCPT 01

Southern Earth Sciences, Inc.

Operator: WRP
Sounding: LCPT-01
Cone Used: DDG0899

CPT Date/Time: 11/11/2005 2:44:07 PM
Location: London Avenue Levee @ Maribeau
Job Number: B05-157



Maximum Depth = 19.03 feet

Depth Increment = 0.164 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

A.IX

GPS Coordinates
N30 00.538' W90 04.167'

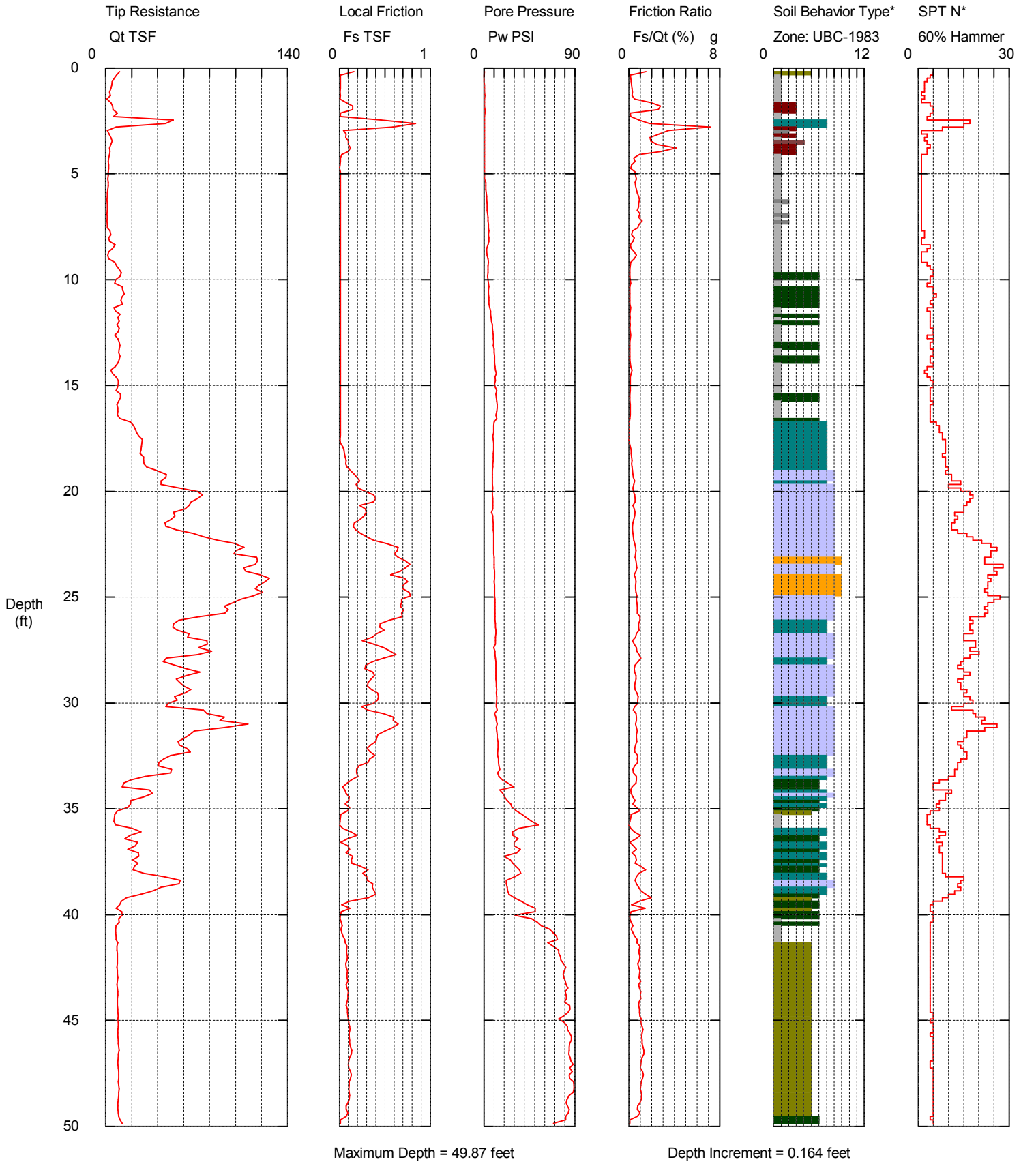
*Soil behavior type and SPT based on data from UBC-1983

Phase II LCPT 02

Southern Earth Sciences, Inc.

Operator: WRP
Sounding: LCPT-02
Cone Used: DDG0899

CPT Date/Time: 11/11/2005 5:07:48 PM
Location: London Avenue Levee @ R.E. Lee
Job Number: B05-157



Maximum Depth = 49.87 feet

Depth Increment = 0.164 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

A.IX

GPS Coordinates
N30 01.236' W90 04.267'

*Soil behavior type and SPT based on data from UBC-1983

17th Street CPT-1A.INT

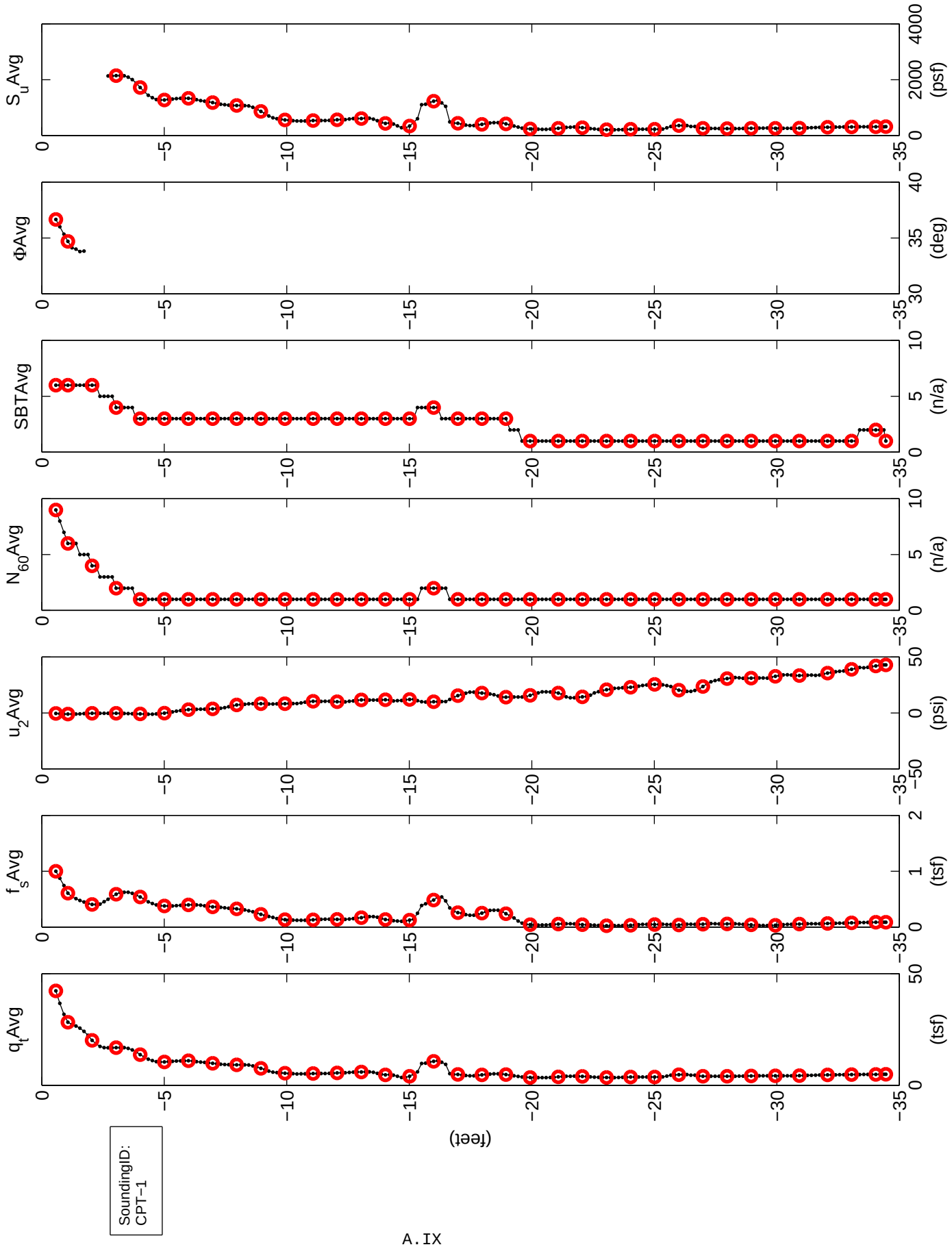
"
"INPUT FILE: 17thstreetCPT-1A.v2.cpt.mat
"File Signature: 17thstreetCPT-1A.cpt.mat[unknown]
"Output Created By SESIexp1, Version 1.01
"Developed By: Dennis O'Meara
"====================================
"
"Interpreter Name: SHS
"
"SUMMARY SHEET
"-------------
"'a' for calculating Qt: 0.800
"Value for Water Table (in m): AS NOTED BELOW - SHOWN BELOW GROUND SURFACE
"Valid Zone Classification based on: Rf
"Missing unit weight to start depth: 18.860
"Method for calculating Su: Nkt
"Define Zone 6 for Sand Parameters? YES
"Method for Friction Angle: Kulhway_Mayne
"Vertical Flow Gradient, i (- up): +0.000
"
"Soil Behavior Type Zone Numbers
"For Rf Zone & Bq Zone Classification
"-----------------------------------
"Zone #1=Sensitive fine grained Zone #7 =Sand with some Silt
"Zone #2=Organic material Zone #8 =Fine sand
"Zone #3=Clay Zone #9 =Sand
"Zone #4=Silty clay Zone #10=Gravelly sand
"Zone #5=Clayey silt Zone #11=Very stiff fine grained *
"Zone #6=Silty sand Zone #12=Sand to clayey sand *
"   * Overconsolidated and/or cemented
"
"NOTE:
"-----
"For soil classification, Rf values > 8 are assumed to be 8.
"
"(Note: --- means Out Of Range)"
"
"Sounding ID: CPT-1A
"Total Depth: 34.94 (ft)
"Nkt: 15.00
"WaterTable: 4.23 (ft)
"
"

"depth (feet)	q_tAvg (tsf)	f_sAvg (tsf)	u_2Avg (psi)	N_60Avg (n/a)	SBTAvg (n/a)	PhiAvg (deg)	S_uAvg (psf)
0.6	42.340	1.002	-0.437	9	6	37	---
1.1	28.314	0.609	-0.934	6	6	35	---
2.1	20.205	0.407	-0.287	4	6	---	---
3.0	16.945	0.591	-0.229	2	4	---	2149
4.0	13.825	0.541	-0.884	1	3	---	1726
5.0	10.500	0.379	-0.190	1	3	---	1275
6.0	11.043	0.398	2.993	1	3	---	1340
7.0	9.940	0.363	3.615	1	3	---	1186
8.0	9.210	0.329	7.204	1	3	---	1081
8.9	7.686	0.231	8.265	1	3	---	870
9.9	5.453	0.135	8.301	1	3	---	565
11.1	5.344	0.133	10.435	1	3	---	542
12.1	5.598	0.140	10.032	1	3	---	569
13.0	6.001	0.171	11.727	1	3	---	615
14.0	4.721	0.139	11.879	1	3	---	437
15.0	4.105	0.126	12.047	1	3	---	348
16.0	10.814	0.487	10.025	2	4	---	1234
17.0	4.941	0.261	15.560	1	3	---	444
18.0	4.704	0.251	17.822	1	3	---	405
18.9	4.890	0.242	14.095	1	3	---	423
19.9	3.580	0.044	15.776	1	1	---	241
21.1	3.829	0.055	17.767	1	1	---	266
22.1	4.045	0.044	14.406	1	1	---	287
23.0	3.569	0.026	20.839	1	1	---	217
24.0	3.763	0.035	23.006	1	1	---	233
25.0	3.781	0.047	25.520	1	1	---	230
26.0	4.819	0.040	20.470	1	1	---	360
27.0	4.155	0.054	23.730	1	1	---	265
28.0	4.122	0.058	30.778	1	1	---	253
29.0	4.258	0.041	31.141	1	1	---	264
29.9	4.323	0.035	32.795	1	1	---	265
30.9	4.408	0.057	33.487	1	1	---	270
32.1	4.713	0.067	35.586	1	1	---	301
33.1	4.851	0.077	39.067	1	1	---	313
34.0	4.971	0.088	42.071	1	2	---	321

17th Street CPT-1A.INT

34.4	5.006	0.087	42.990	1	1	---	322

17th Street CPT-1A.SESIAvgEXP



17th Street CPT2A.INT

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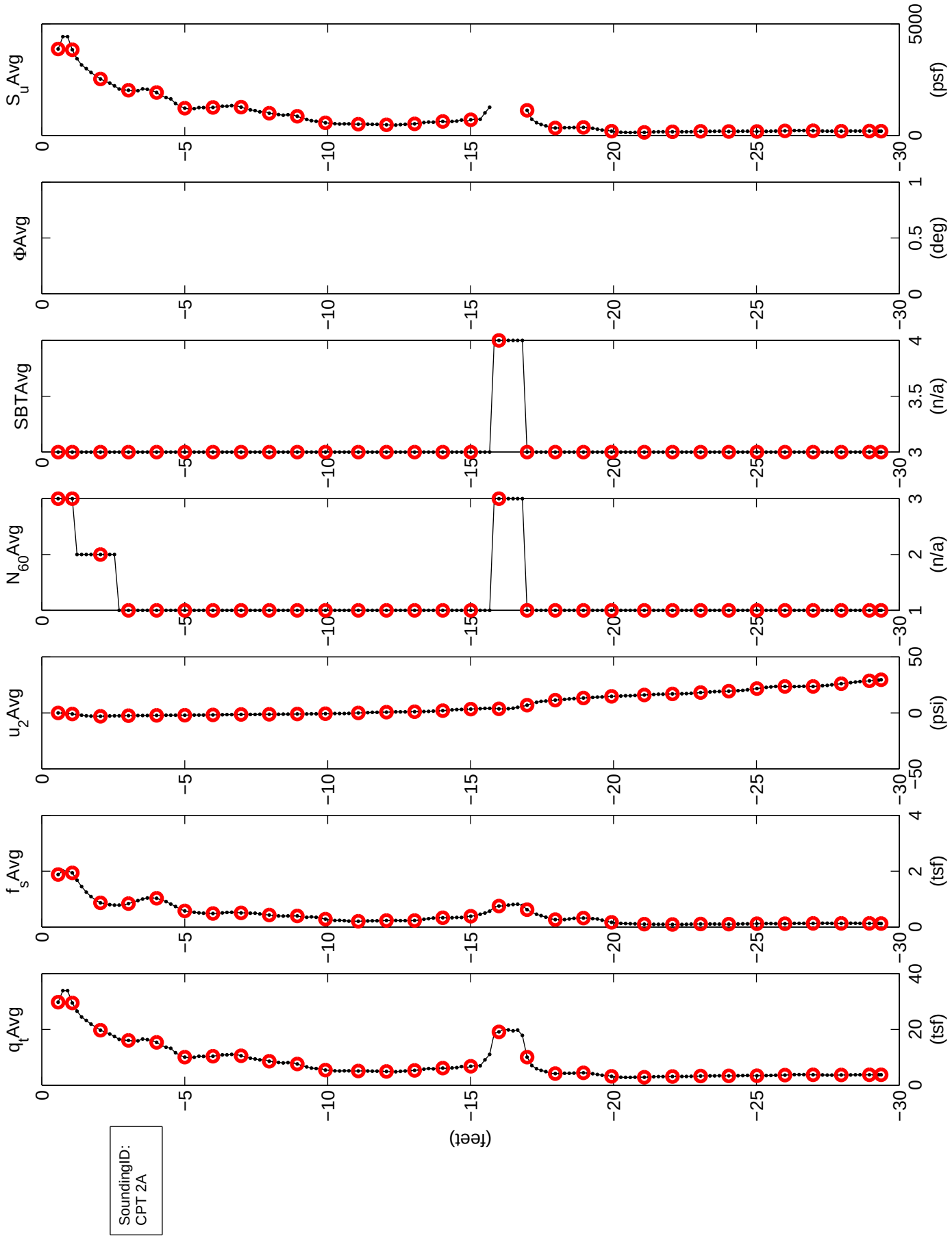
"File Signature: 17thstreetCPT 2A.cpt.mat[unknown]
"Output Created By SESIexp1, Version 1.01
"Developed By: Dennis O'Meara
"====================================
"
"Interpreter Name: SHS
"
"SUMMARY SHEET
"-------------
"'a' for calculating Qt: 0.800
"Value for Water Table (in m): AS NOTED BELOW - SHOWN BELOW GROUND SURFACE
"Valid Zone Classification based on: Rf
"Missing unit weight to start depth: 18.860
"Method for calculating Su: Nkt
"Define Zone 6 for Sand Parameters? YES
"Method for Friction Angle: Kulhway_Mayne
"Vertical Flow Gradient, i (- up): +0.000
"
"Soil Behavior Type Zone Numbers
"For Rf Zone & Bq Zone Classification
"-----------------------------------
"Zone #1=Sensitive fine grained Zone #7 =Sand with some Silt
"Zone #2=Organic material Zone #8 =Fine sand
"Zone #3=Clay Zone #9 =Sand
"Zone #4=Silty clay Zone #10=Gravelly sand
"Zone #5=Clayey silt Zone #11=Very stiff fine grained *
"Zone #6=Silty sand Zone #12=Sand to clayey sand *
"   * Overconsolidated and/or cemented
"
"NOTE:
"-----
"For soil classification, Rf values > 8 are assumed to be 8.
"
"(Note: --- means Out Of Range)"
"
"Sounding ID: CPT 2A
"Total Depth: 29.86 (ft)
"Nkt: 15.00
"WaterTable: 4.23 (ft)
"
"

"depth (feet)	q_tAvg (tsf)	f_sAvg (tsf)	u_2Avg (psi)	N_60Avg (n/a)	SBTAvg (n/a)	PhiAvg (deg)	S_uAvg (psf)
0.6	29.773	1.885	0.033	3	3	---	3878
1.1	29.535	1.942	-0.962	3	3	---	3844
2.1	19.771	0.872	-3.045	2	3	---	2535
3.0	16.086	0.843	-2.428	1	3	---	2035
4.0	15.380	1.031	-2.177	1	3	---	1934
5.0	10.129	0.583	-1.998	1	3	---	1227
6.0	10.448	0.489	-1.741	1	3	---	1261
7.0	10.622	0.510	-1.422	1	3	---	1278
8.0	8.660	0.434	-1.208	1	3	---	1008
8.9	7.657	0.401	-1.002	1	3	---	868
9.9	5.498	0.285	-0.711	1	3	---	572
11.1	5.148	0.211	-0.036	1	3	---	517
12.1	4.977	0.235	0.746	1	3	---	487
13.0	5.356	0.236	1.115	1	3	---	530
14.0	6.211	0.333	2.083	1	3	---	638
15.0	6.828	0.392	3.459	1	3	---	712
16.0	19.152	0.755	3.645	3	4	---	---
17.0	10.084	0.622	6.943	1	3	---	1132
18.0	4.246	0.271	11.413	1	3	---	345
18.9	4.484	0.325	13.345	1	3	---	372
19.9	3.262	0.170	14.773	1	3	---	201
21.1	2.892	0.104	16.097	1	3	---	145
22.1	3.175	0.097	16.956	1	3	---	174
23.0	3.356	0.113	18.169	1	3	---	191
24.0	3.401	0.104	19.485	1	3	---	189
25.0	3.448	0.122	21.767	1	3	---	190
26.0	3.697	0.123	23.609	1	3	---	215
27.0	3.782	0.131	23.691	1	3	---	218
28.0	3.715	0.132	26.145	1	3	---	203
29.0	3.812	0.131	28.704	1	3	---	209
29.4	3.791	0.129	29.656	1	3	---	203

17th Street CPT 2A.SESIAvgEXP



17th Street CPT3.INT

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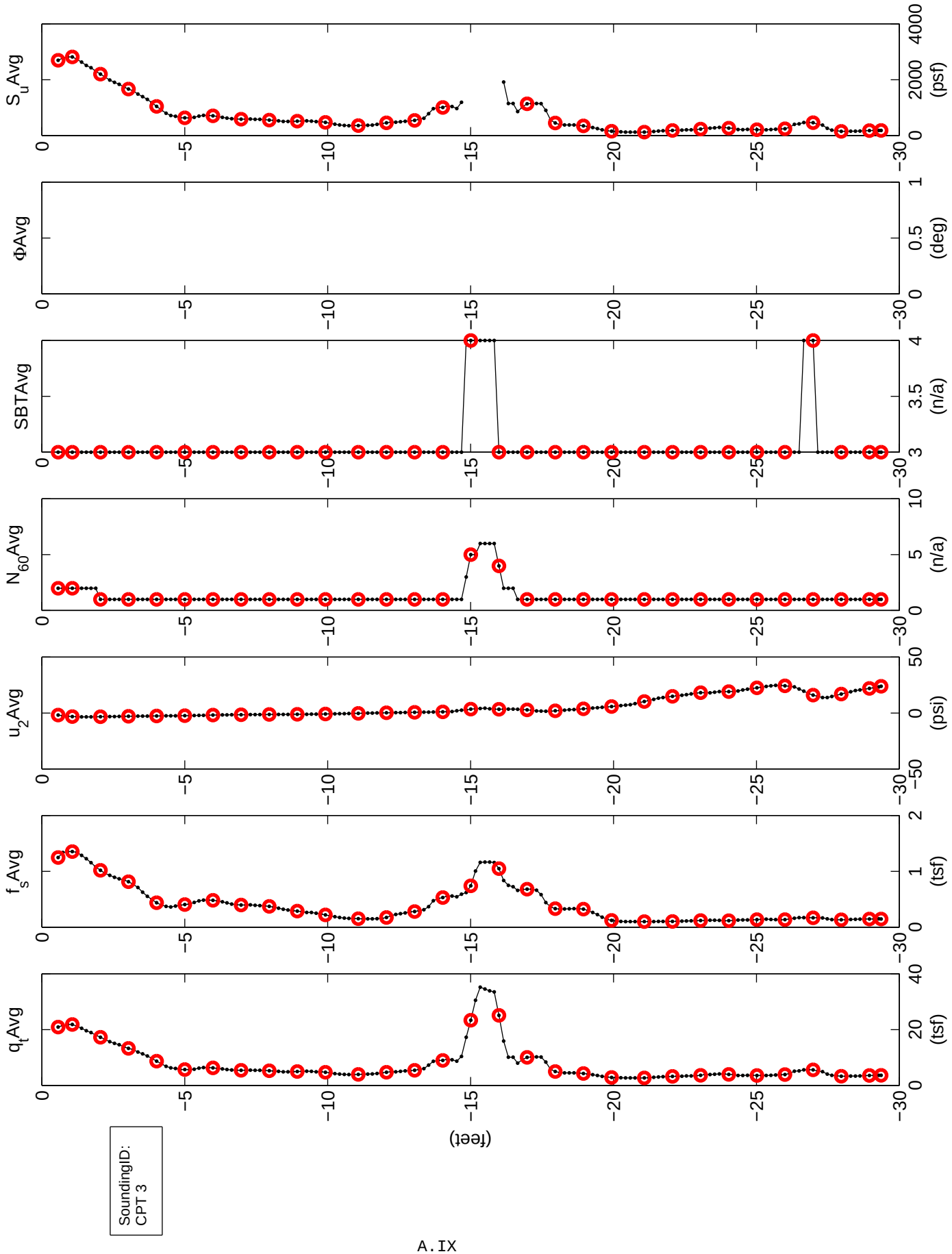
"File Signature: 17thstreetCPT 3.cpt.mat[unknown]
"Output Created By SESIexp1, Version 1.01
"Developed By: Dennis O'Meara
"====================================
"
"Interpreter Name: SHS
"
"SUMMARY SHEET
"-------------
"'a' for calculating Qt: 0.800
"Value for Water Table (in m): AS NOTED BELOW - SHOWN BELOW GROUND SURFACE
"Valid Zone Classification based on: Rf
"Missing unit weight to start depth: 18.860
"Method for calculating Su: Nkt
"Define Zone 6 for Sand Parameters? YES
"Method for Friction Angle: Kulhway_Mayne
"Vertical Flow Gradient, i (- up): +0.000
"
"Soil Behavior Type Zone Numbers
"For Rf Zone & Bq Zone Classification
"-----------------------------------
"Zone #1=Sensitive fine grained Zone #7 =Sand with some Silt
"Zone #2=Organic material Zone #8 =Fine sand
"Zone #3=Clay Zone #9 =Sand
"Zone #4=Silty clay Zone #10=Gravelly sand
"Zone #5=Clayey silt Zone #11=Very stiff fine grained *
"Zone #6=Silty sand Zone #12=Sand to clayey sand *
" * Overconsolidated and/or cemented
"
"NOTE:
"-----
"For soil classification, Rf values > 8 are assumed to be 8.
"
"(Note: --- means Out Of Range)"
"
"Sounding ID: CPT 3
"Total Depth: 29.86 (ft)
"Nkt: 15.00
"WaterTable: 4.23 (ft)
"
"

"depth (feet)	q_tAvg (tsf)	f_sAvg (tsf)	u_2Avg (psi)	N_60Avg (n/a)	SBTAvg (n/a)	PhiAvg (deg)	S_uAvg (psf)
0.6	20.949	1.249	-1.804	2	3	---	2702
1.1	21.867	1.356	-3.063	2	3	---	2821
2.1	17.290	1.020	-3.237	1	3	---	2203
3.0	13.348	0.815	-2.879	1	3	---	1670
4.0	8.738	0.441	-2.575	1	3	---	1048
5.0	5.723	0.408	-2.310	1	3	---	638
6.0	6.326	0.483	-1.801	1	3	---	712
7.0	5.470	0.400	-1.465	1	3	---	590
8.0	5.302	0.374	-1.266	1	3	---	561
8.9	5.045	0.291	-1.099	1	3	---	520
9.9	4.761	0.222	-0.824	1	3	---	475
11.1	3.995	0.153	-0.232	1	3	---	364
12.1	4.759	0.175	0.234	1	3	---	459
13.0	5.463	0.282	0.648	1	3	---	546
14.0	9.000	0.533	1.140	1	3	---	1011
15.0	23.431	0.744	3.480	5	4	---	---
16.0	25.144	1.051	3.422	4	3	---	---
17.0	10.130	0.683	2.865	1	3	---	1139
18.0	5.049	0.333	2.014	1	3	---	455
18.9	4.326	0.325	3.798	1	3	---	353
19.9	2.927	0.123	5.899	1	3	---	159
21.1	2.756	0.102	10.358	1	3	---	127
22.1	3.281	0.104	14.946	1	3	---	190
23.0	3.687	0.120	18.201	1	3	---	237
24.0	4.005	0.119	19.197	1	3	---	271
25.0	3.651	0.139	22.580	1	3	---	217
26.0	3.948	0.137	24.412	1	3	---	250
27.0	5.642	0.171	16.058	1	4	---	469
28.0	3.361	0.134	17.074	1	3	---	156
29.0	3.619	0.148	22.063	1	3	---	184
29.4	3.670	0.149	23.947	1	3	---	187

17th Street CPT 3.SESIAvgEXP



IHNCCPT-1_INT

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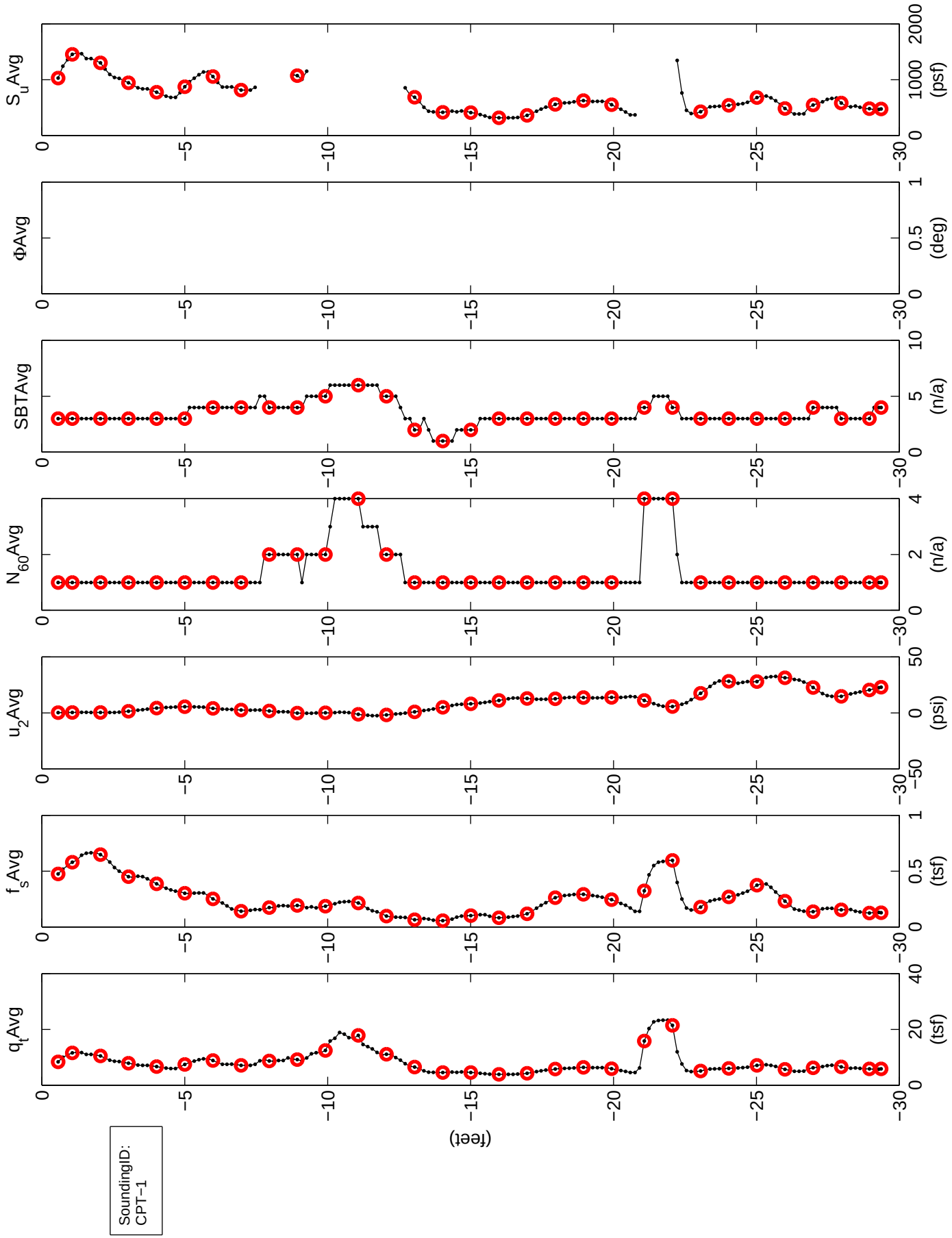
"File Signature: IHNCCPT-1.cpt.mat[unknown]
"Output Created By SESIexp1, Version 1.01
"Developed By: Dennis O'Meara
"====================================
"
"Interpreter Name: SHS
"
"SUMMARY SHEET
"-------------
"'a' for calculating Qt: 0.800
"Value for Water Table (in m): AS NOTED BELOW - SHOWN BELOW GROUND SURFACE
"Valid Zone Classification based on: Rf
"Missing unit weight to start depth: 18.860
"Method for calculating Su: Nkt
"Define Zone 6 for Sand Parameters? YES
"Method for Friction Angle: Kulhway_Mayne
"Vertical Flow Gradient, i (- up): +0.000
"
"Soil Behavior Type Zone Numbers
"For Rf Zone & Bq Zone Classification
"-----------------------------------
"Zone #1=Sensitive fine grained Zone #7 =Sand with some Silt
"Zone #2=Organic material Zone #8 =Fine sand
"Zone #3=Clay Zone #9 =Sand
"Zone #4=Silty clay Zone #10=Gravelly sand
"Zone #5=Clayey silt Zone #11=Very stiff fine grained *
"Zone #6=Silty sand Zone #12=Sand to clayey sand *
" * Overconsolidated and/or cemented
"
"NOTE:
"-----
"For soil classification, Rf values > 8 are assumed to be 8.
"
"(Note: --- means Out Of Range)"
"
"Sounding ID: CPT-1B
"Total Depth: 29.86 (ft)
"Nkt: 15.00
"WaterTable: 4.23 (ft)
"
"

"depth (feet)	q_tAvg (tsf)	f_sAvg (tsf)	u_2Avg (psi)	N_60Avg (n/a)	SBTAvg (n/a)	PhiAvg (deg)	S_uAvg (psf)
0.6	8.427	0.477	0.293	1	3	---	1032
1.1	11.641	0.582	0.457	1	3	---	1458
2.1	10.544	0.649	0.426	1	3	---	1304
3.0	7.932	0.452	1.455	1	3	---	947
4.0	6.721	0.388	4.301	1	3	---	779
5.0	7.490	0.303	5.576	1	3	---	875
6.0	8.917	0.252	4.115	1	4	---	1057
7.0	7.166	0.142	2.652	1	4	---	816
8.0	8.734	0.174	1.793	2	4	---	---
8.9	9.215	0.195	-0.168	2	4	---	1075
9.9	12.482	0.187	0.109	2	5	---	---
11.1	17.915	0.216	-1.181	4	6	---	---
12.1	11.149	0.099	-1.707	2	5	---	---
13.0	6.549	0.066	0.970	1	2	---	688
14.0	4.587	0.059	5.074	1	1	---	419
15.0	4.589	0.103	8.191	1	2	---	412
16.0	3.962	0.084	11.137	1	3	---	320
17.0	4.345	0.119	12.893	1	3	---	364
18.0	5.857	0.264	12.704	1	3	---	559
18.9	6.434	0.293	13.771	1	3	---	629
19.9	5.943	0.247	13.825	1	3	---	556
21.1	15.891	0.325	11.189	4	4	---	---
22.1	21.476	0.597	5.660	4	4	---	---
23.0	5.172	0.179	17.552	1	3	---	430
24.0	6.079	0.271	28.261	1	3	---	543
25.0	7.180	0.375	27.965	1	3	---	682
26.0	5.761	0.232	31.384	1	3	---	486
27.0	6.288	0.139	22.755	1	4	---	549
28.0	6.602	0.154	14.791	1	3	---	584
29.0	5.901	0.126	20.517	1	3	---	483
29.4	5.893	0.128	22.960	1	4	---	479

IHNCCPT-1.SESIAvgExp



IHNCCPT-2.INT

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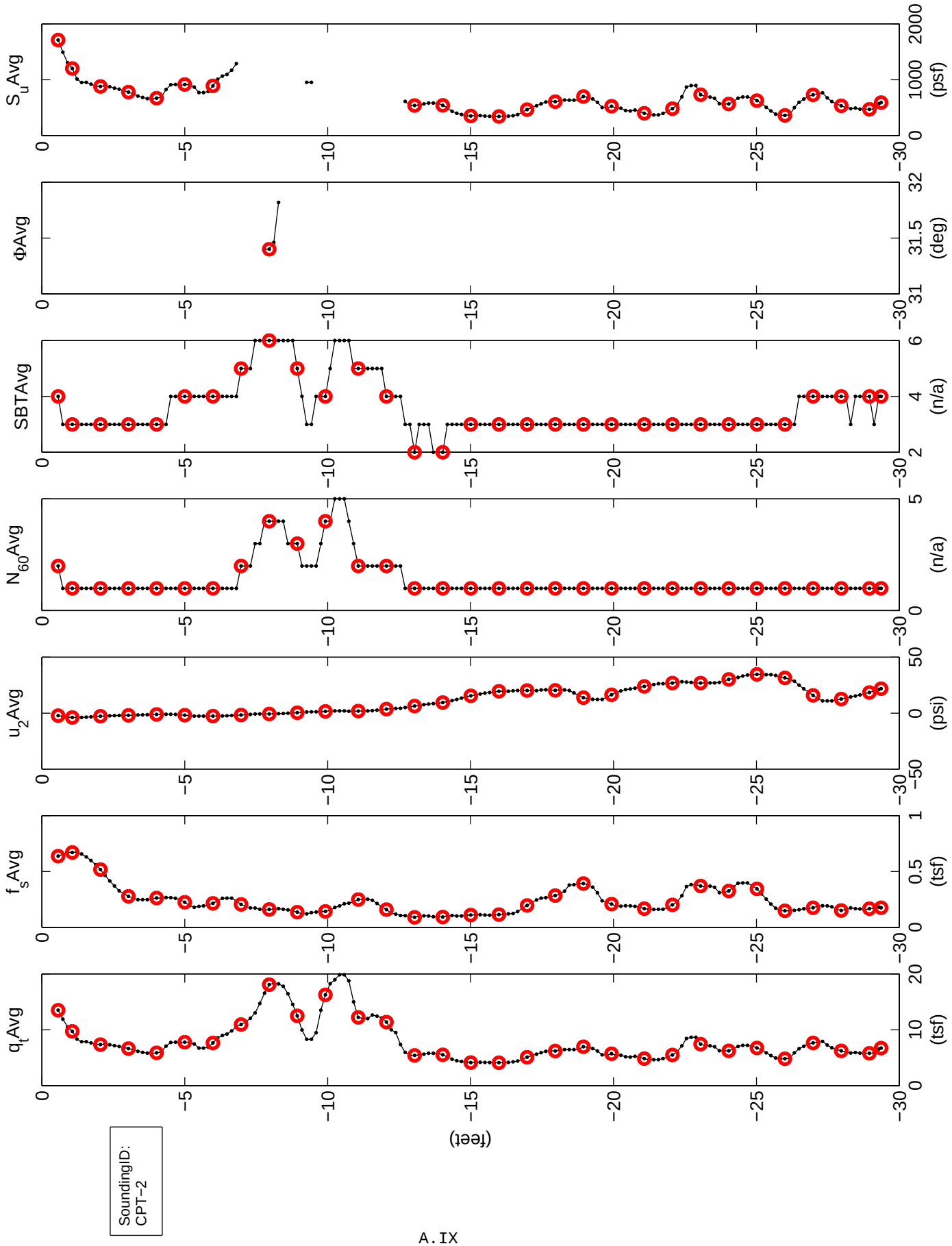
"Output Created By SESIexp1, Version 1.01
"Developed By: Dennis O'Meara
"====================================
"
"Interpreter Name: SHS
"
"SUMMARY SHEET
"-------------
"'a' for calculating Qt: 0.800
"Value for Water Table (in m): AS NOTED BELOW - SHOWN BELOW GROUND SURFACE
"Valid Zone Classification based on: Rf
"Missing unit weight to start depth: 18.860
"Method for calculating Su: Nkt
"Define Zone 6 for Sand Parameters? YES
"Method for Friction Angle: Kulhway_Mayne
"Vertical Flow Gradient, i (- up): +0.000
"
"Soil Behavior Type Zone Numbers
"For Rf Zone & Bq Zone Classification
"-----------------------------------
"Zone #1=Sensitive fine grained Zone #7 =Sand with some Silt
"Zone #2=Organic material Zone #8 =Fine sand
"Zone #3=Clay Zone #9 =Sand
"Zone #4=Silty clay Zone #10=Gravelly sand
"Zone #5=Clayey silt Zone #11=Very stiff fine grained *
"Zone #6=Silty sand Zone #12=Sand to clayey sand *
"   * Overconsolidated and/or cemented
"
"NOTE:
"-----
"For soil classification, Rf values > 8 are assumed to be 8.
"
"(Note: --- means Out Of Range)"
"
"Sounding ID: CPT-2
"Total Depth: 29.86 (ft)
"Nkt: 15.00
"WaterTable: 4.23 (ft)
"
"

"depth (feet)	q_tAvg (tsf)	f_sAvg (tsf)	u_2Avg (psi)	N_60Avg (n/a)	SBTAvg (n/a)	PhiAvg (deg)	S_uAvg (psf)
0.6	13.533	0.638	-2.117	2	4	---	1713
1.1	9.742	0.671	-3.869	1	3	---	1203
2.1	7.383	0.519	-2.736	1	3	---	882
3.0	6.672	0.279	-1.840	1	3	---	780
4.0	5.909	0.263	-1.114	1	3	---	671
5.0	7.808	0.225	-1.831	1	4	---	917
6.0	7.677	0.216	-2.564	1	4	---	892
7.0	10.956	0.207	-1.655	2	5	---	---
8.0	18.099	0.162	-0.615	4	6	31	---
8.9	12.523	0.136	0.466	3	5	---	---
9.9	16.272	0.144	1.644	4	4	---	---
11.1	12.256	0.250	1.886	2	5	---	---
12.1	11.407	0.161	3.679	2	4	---	---
13.0	5.446	0.091	6.382	1	2	---	541
14.0	5.534	0.093	9.555	1	2	---	545
15.0	4.167	0.111	15.575	1	3	---	354
16.0	4.139	0.115	19.558	1	3	---	343
17.0	5.116	0.197	20.270	1	3	---	466
18.0	6.225	0.285	20.404	1	3	---	608
18.9	6.980	0.392	13.908	1	3	---	702
19.9	5.732	0.209	16.495	1	3	---	527
21.1	4.862	0.168	23.964	1	3	---	402
22.1	5.530	0.202	26.943	1	3	---	484
23.0	7.456	0.372	26.931	1	3	---	735
24.0	6.256	0.326	30.238	1	3	---	567
25.0	6.773	0.345	34.636	1	3	---	629
26.0	4.831	0.149	31.555	1	3	---	363
27.0	7.662	0.176	15.894	1	4	---	731
28.0	6.253	0.152	12.715	1	4	---	537
29.0	5.824	0.169	18.570	1	4	---	472
29.4	6.746	0.176	21.886	1	4	---	592

IHNCCPT-2.SESIAvgExp



IHNCCPT-3.INT

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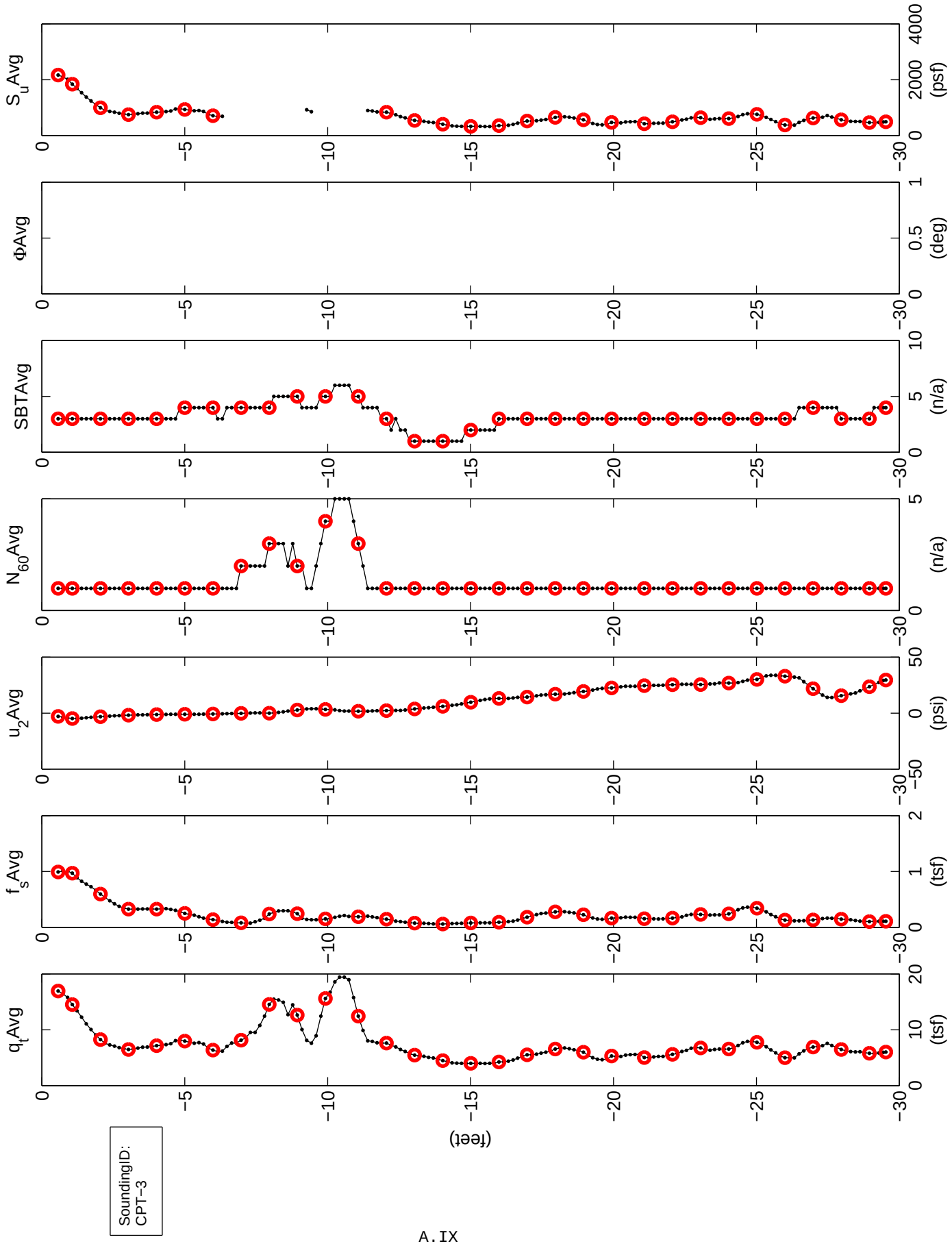
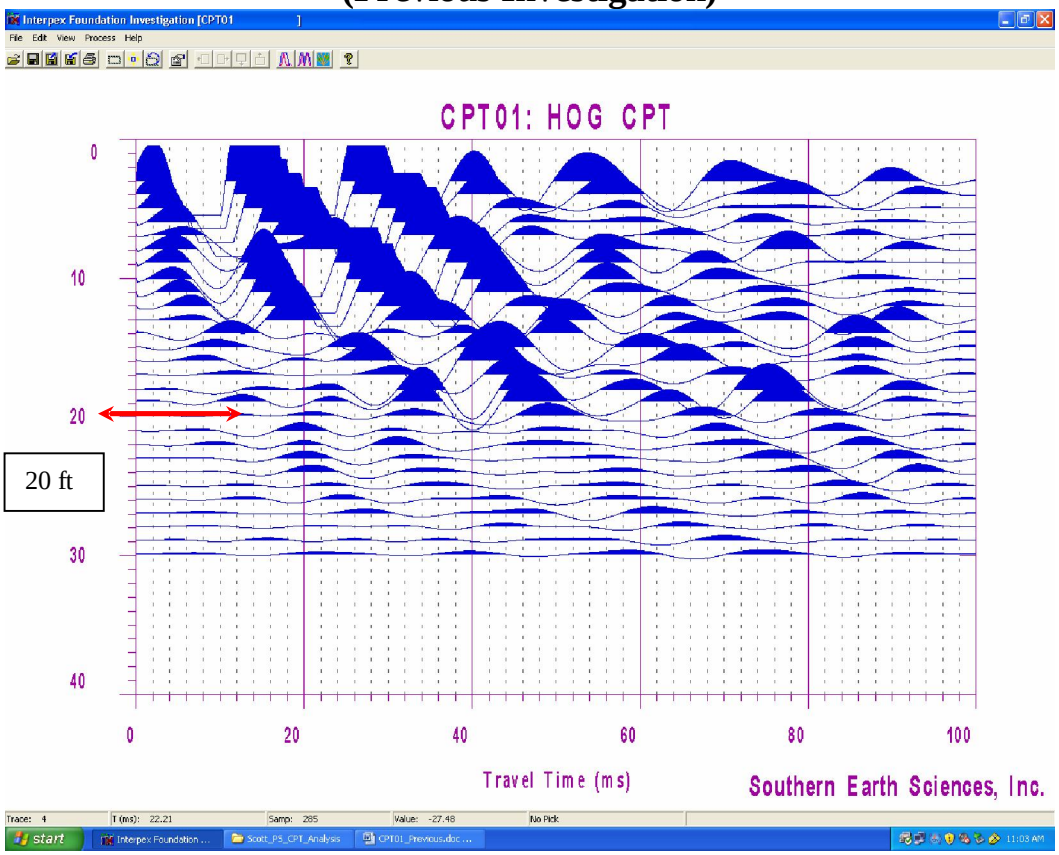
"File Signature: IHNCCPT-3.cpt.mat[unknown]
"Output Created By SESIexp1, Version 1.01
"Developed By: Dennis O'Meara
"====================================
"
"Interpreter Name: SHS
"
"SUMMARY SHEET
"-------------
"'a' for calculating Qt: 0.800
"Value for Water Table (in m): AS NOTED BELOW - SHOWN BELOW GROUND SURFACE
"Valid Zone Classification based on: Rf
"Missing unit weight to start depth: 18.860
"Method for calculating Su: Nkt
"Define Zone 6 for Sand Parameters? YES
"Method for Friction Angle: Kulhway_Mayne
"Vertical Flow Gradient, i (- up): +0.000
"
"Soil Behavior Type Zone Numbers
"For Rf Zone & Bq Zone Classification
"-----------------------------------
"Zone #1=Sensitive fine grained Zone #7 =Sand with some Silt
"Zone #2=Organic material Zone #8 =Fine sand
"Zone #3=Clay Zone #9 =Sand
"Zone #4=Silty clay Zone #10=Gravelly sand
"Zone #5=Clayey silt Zone #11=Very stiff fine grained *
"Zone #6=Silty sand Zone #12=Sand to clayey sand *
"   * Overconsolidated and/or cemented
"
"NOTE:
"-----
"For soil classification, Rf values > 8 are assumed to be 8.
"
"(Note: --- means Out Of Range)"
"
"Sounding ID: CPT-3
"Total Depth: 30.02 (ft)
"Nkt: 15.00
"WaterTable: 4.23 (ft)
"
"

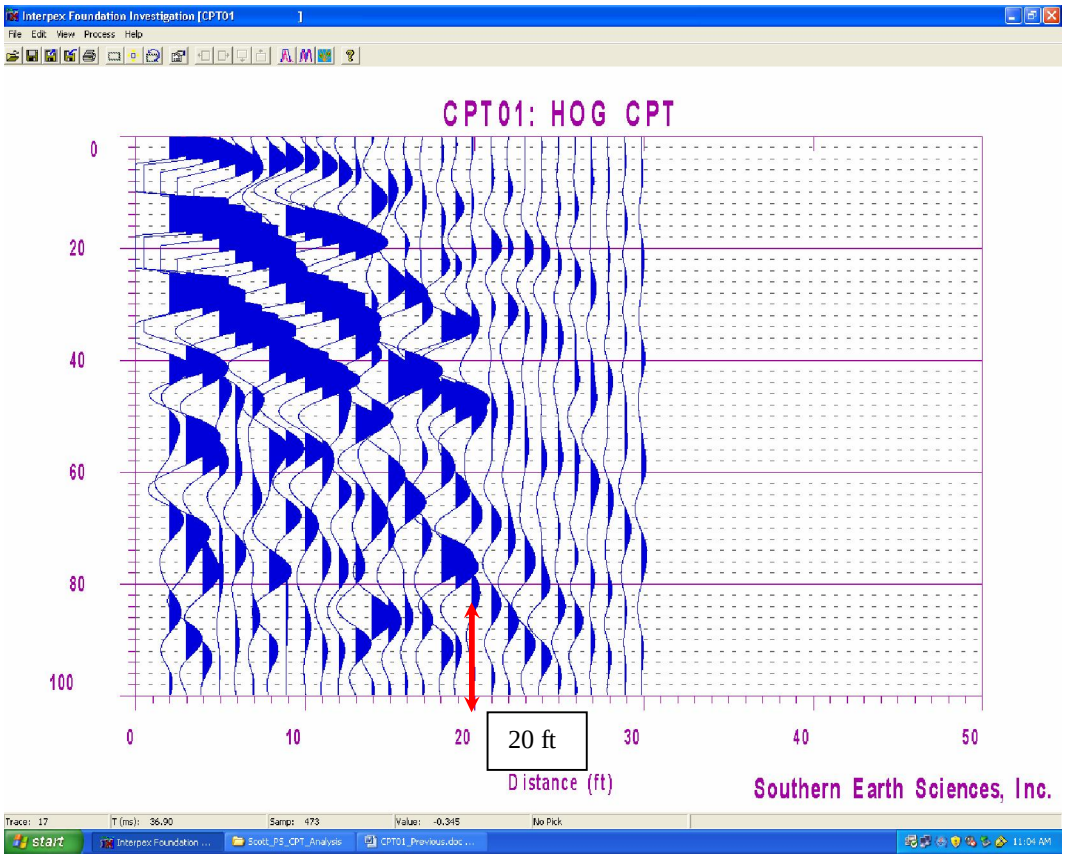
"depth (feet)	q_tAvg (tsf)	f_sAvg (tsf)	u_2Avg (psi)	N_60Avg (n/a)	SBTAvg (n/a)	PhiAvg (deg)	S_uAvg (psf)
0.6	16.964	0.993	-2.879	1	3	---	2171
1.1	14.535	0.972	-4.711	1	3	---	1843
2.1	8.263	0.599	-3.152	1	3	---	999
3.0	6.498	0.329	-1.810	1	3	---	757
4.0	7.180	0.329	-1.208	1	3	---	840
5.0	7.992	0.252	-0.995	1	4	---	941
6.0	6.377	0.141	-0.722	1	4	---	718
7.0	8.173	0.084	-0.127	2	4	---	---
8.0	14.576	0.242	0.020	3	4	---	---
8.9	12.650	0.247	2.845	2	5	---	---
9.9	15.624	0.155	3.413	4	5	---	---
11.1	12.461	0.193	1.815	3	5	---	---
12.1	7.650	0.150	2.323	1	3	---	843
13.0	5.502	0.079	3.742	1	1	---	548
14.0	4.522	0.062	6.151	1	1	---	410
15.0	4.011	0.080	9.860	1	2	---	335
16.0	4.258	0.093	13.171	1	3	---	361
17.0	5.551	0.184	14.394	1	3	---	525
18.0	6.567	0.280	17.018	1	3	---	655
18.9	6.001	0.227	19.417	1	3	---	571
19.9	5.329	0.167	22.643	1	3	---	475
21.1	5.060	0.156	24.563	1	3	---	428
22.1	5.628	0.166	25.504	1	3	---	498
23.0	6.782	0.232	25.517	1	3	---	645
24.0	6.590	0.243	26.972	1	3	---	612
25.0	7.791	0.345	30.230	1	3	---	765
26.0	5.019	0.133	33.042	1	3	---	387
27.0	6.948	0.134	21.909	1	4	---	637
28.0	6.505	0.149	15.670	1	3	---	572
29.0	5.817	0.105	23.666	1	3	---	471
29.5	6.044	0.110	29.640	1	4	---	498

IHNCCPT-3.SESIAvgExp



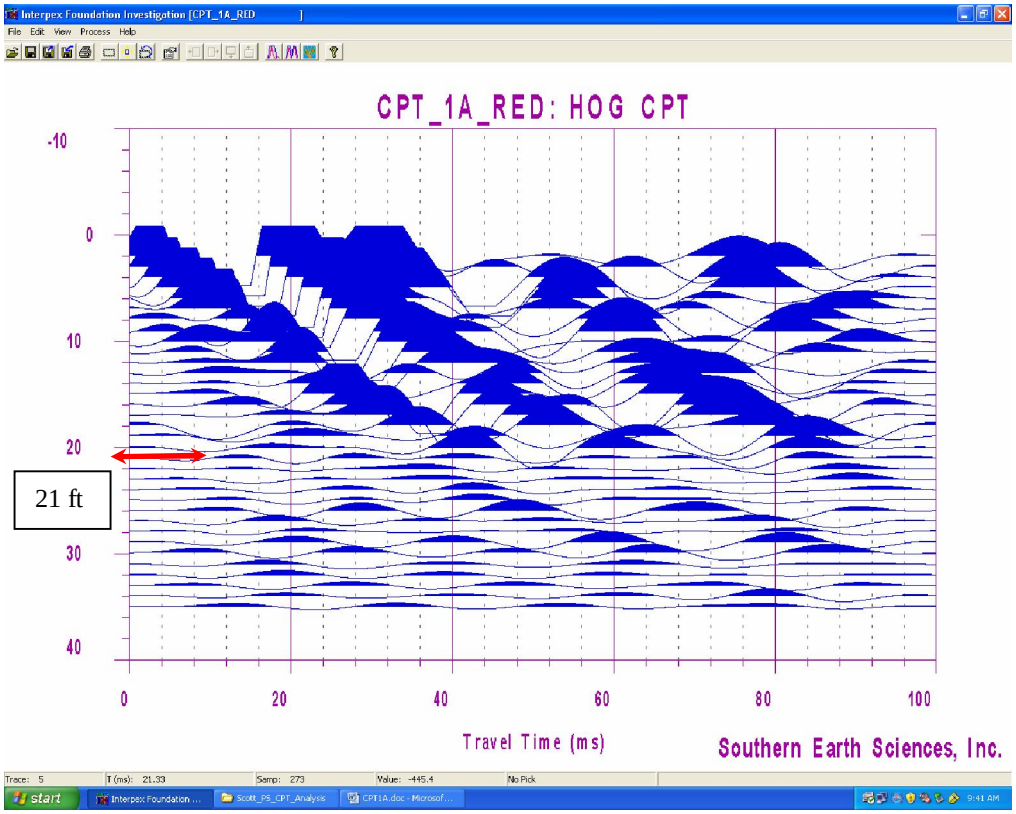
17th Street Canal Levee
PS/CPT Data Analysis
(Previous Investigation)

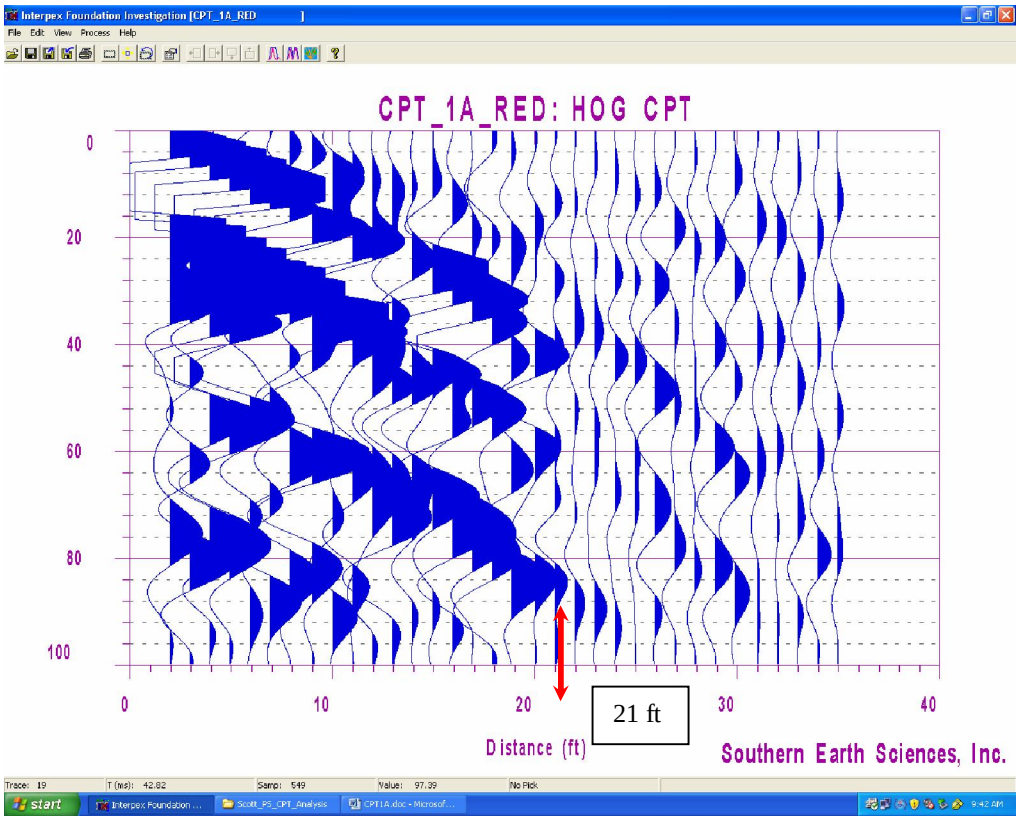




GPS Coordinates: N30°00.890' W90°07.293'; approximately 400 ft south of breach. Field-testing performed on Nov. 10, 2005.

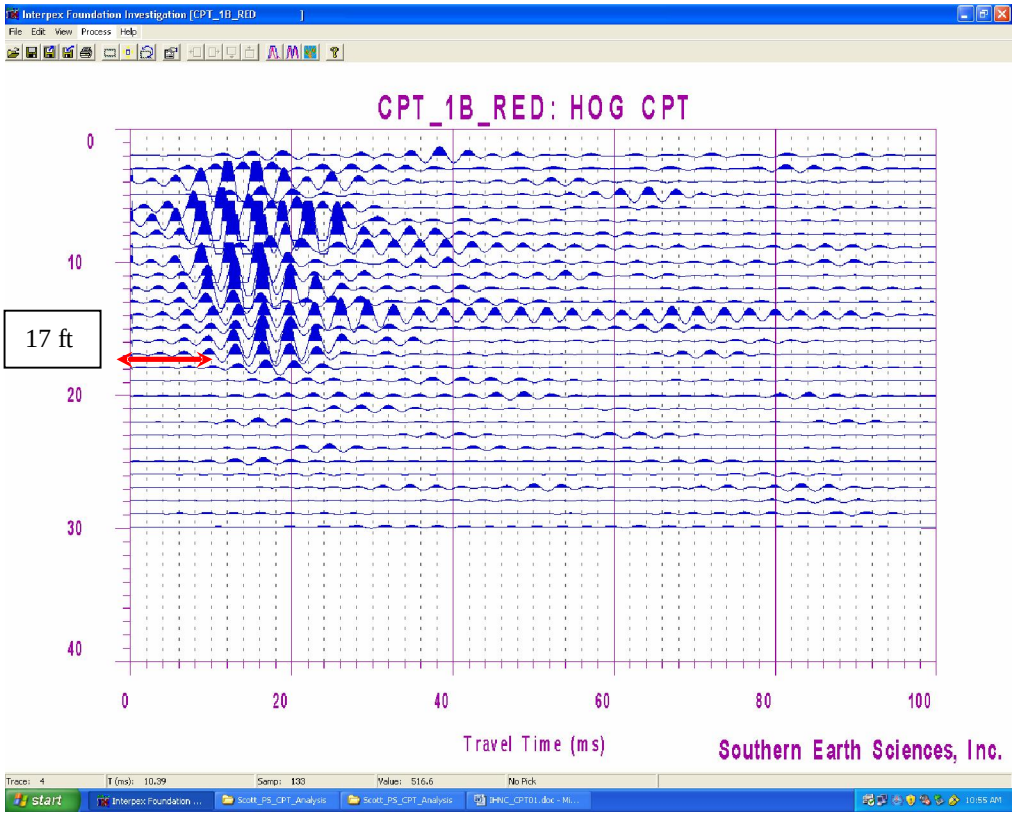
17th Street Canal Levee
PS/CPT Data Analysis

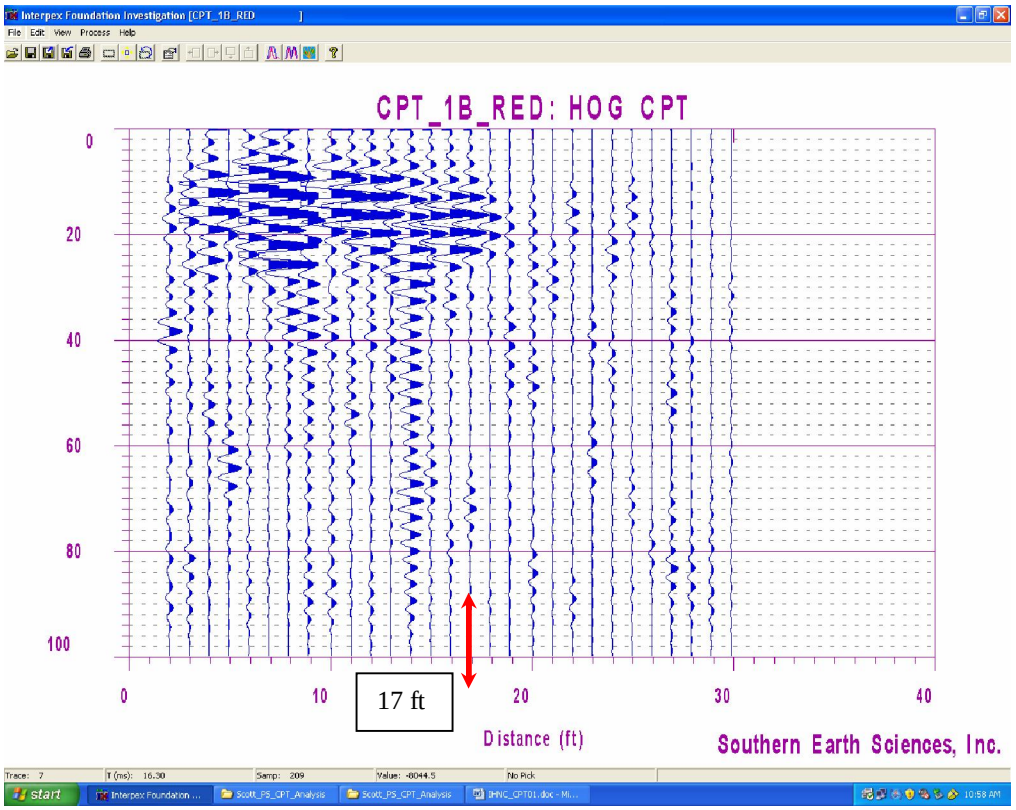




Sta. 20+78, south end of breach. Field-testing performed on Dec. 22, 2005.

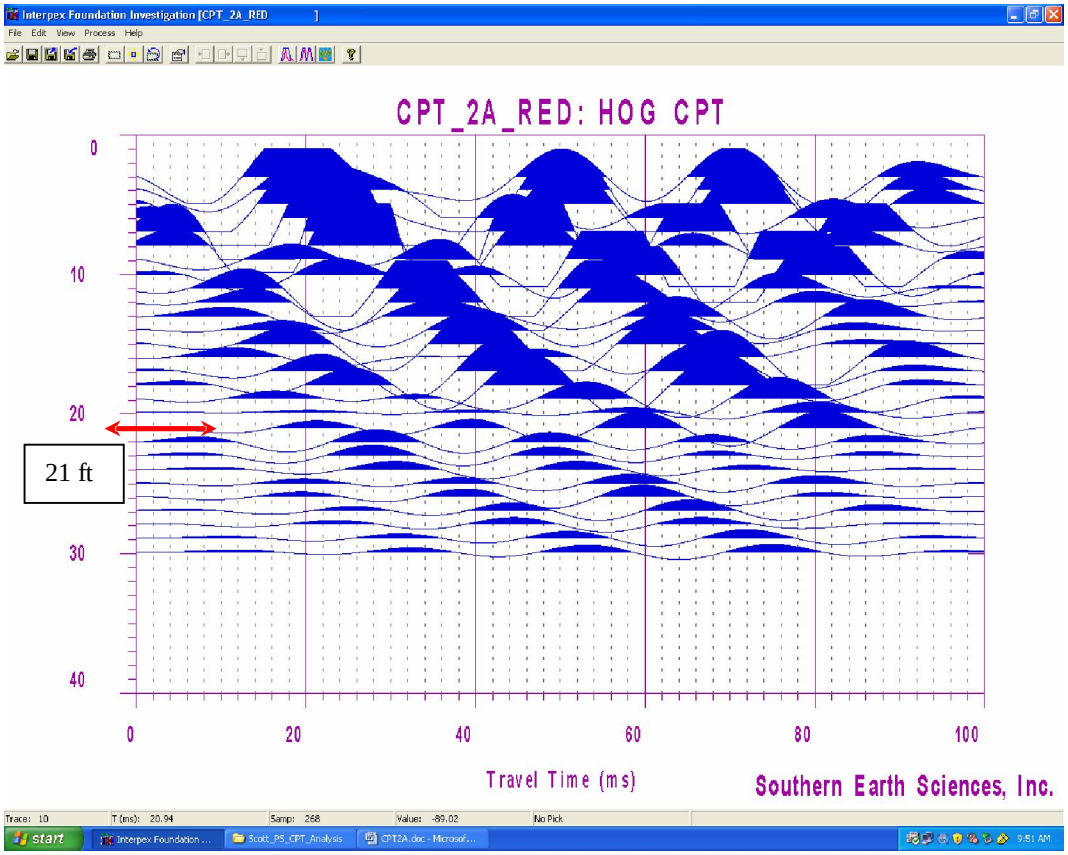
IHNC
PS/CPT Data Analysis

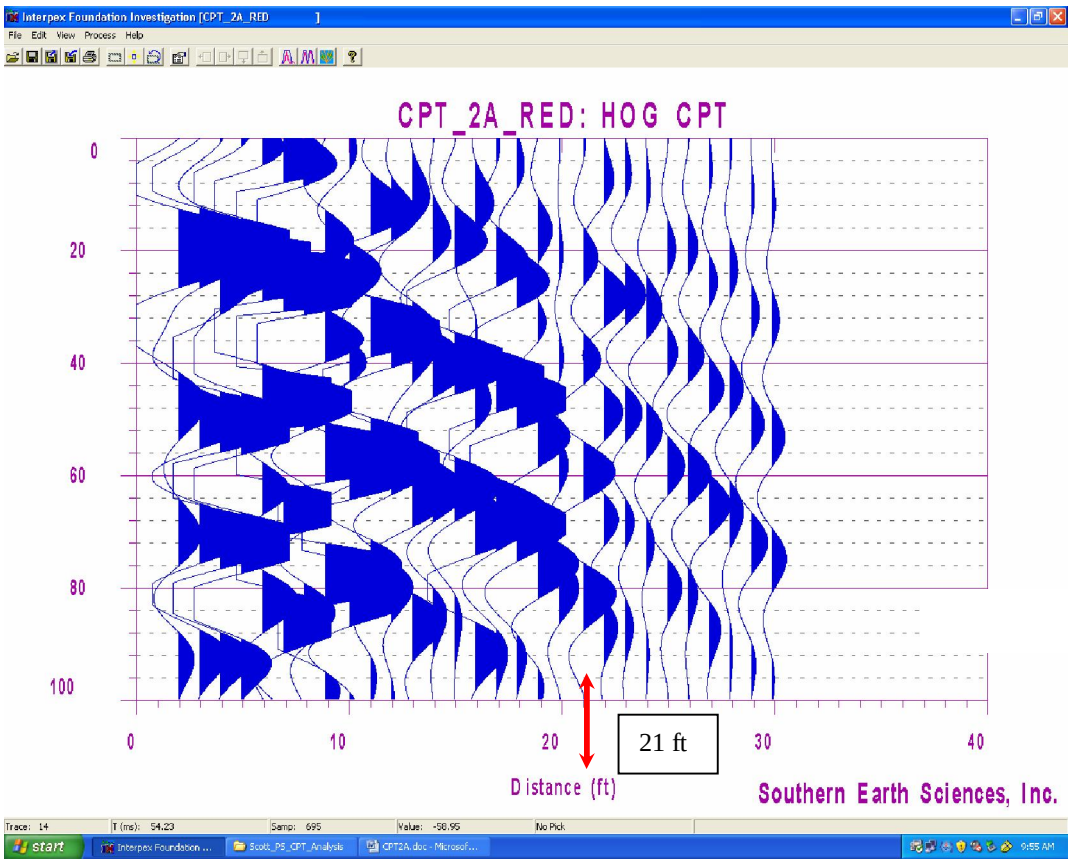




Sta. 17+11. Field-testing performed on Dec. 21, 2005.

17th Street Canal Levee
PS/CPT Data Analysis

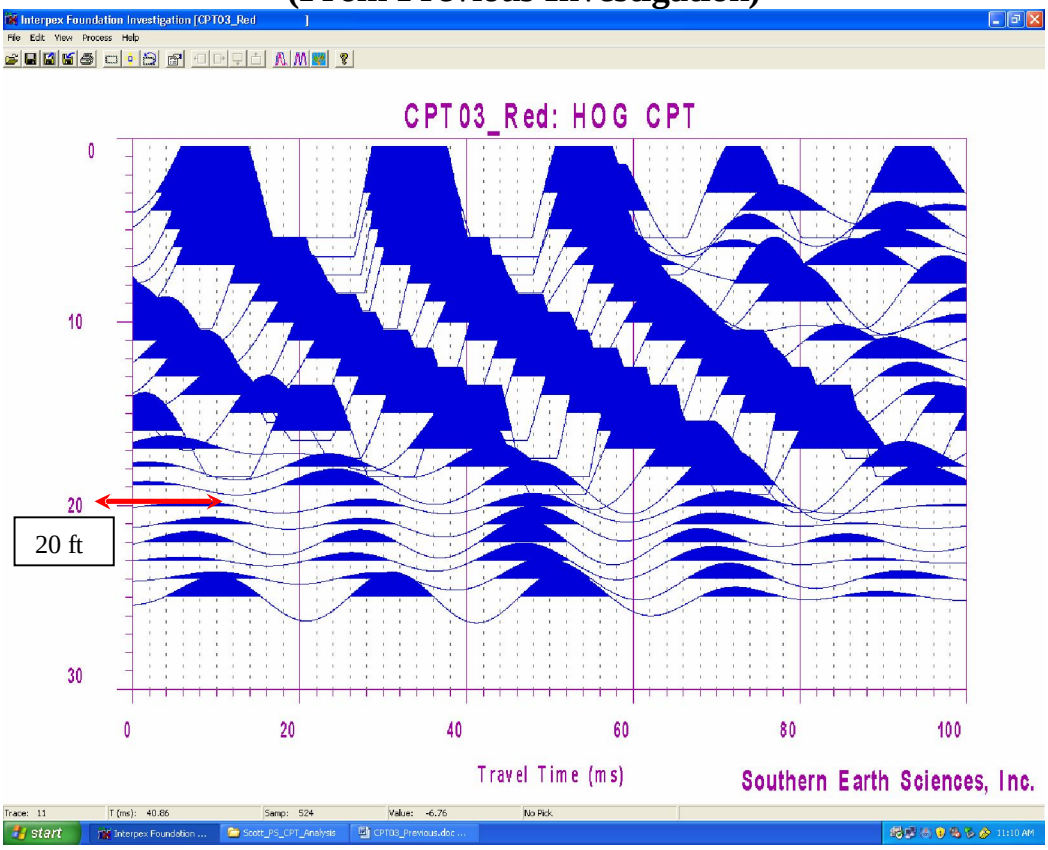


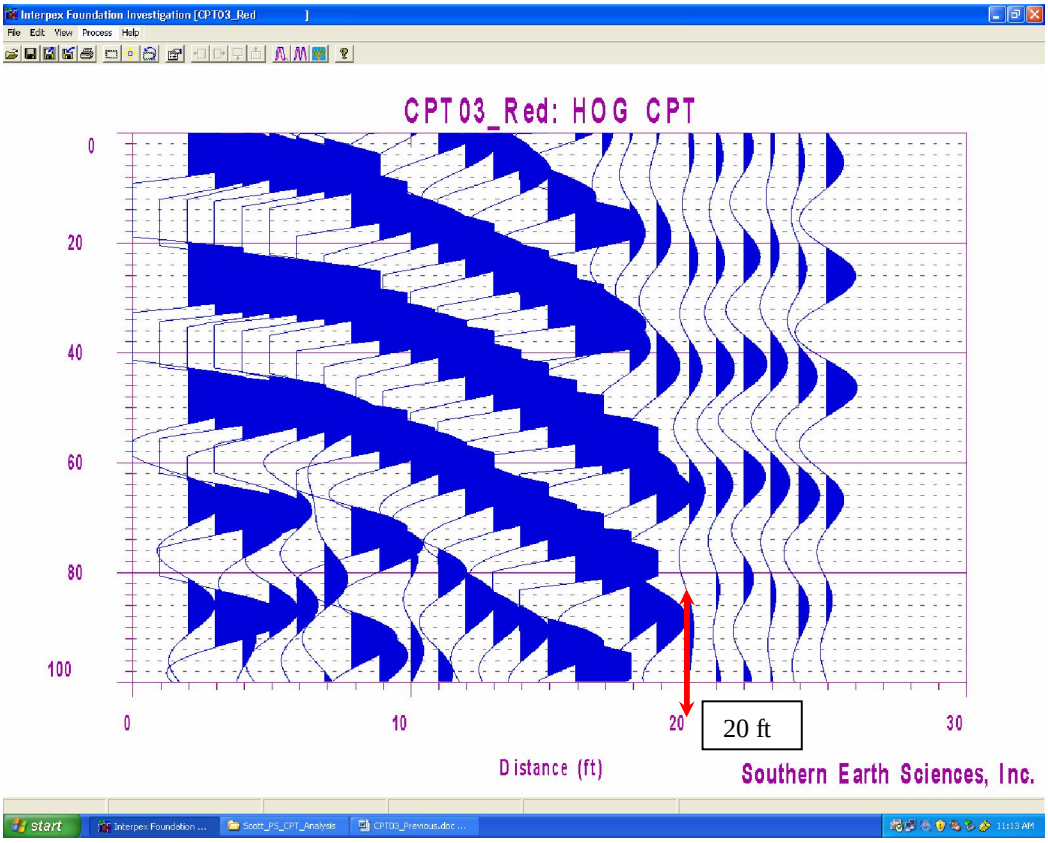


Sta. 20+77 (approx. 1 ft from CPT-1A). Field testing performed on Dec. 22, 2005.

NDT Results PS_CPT_3

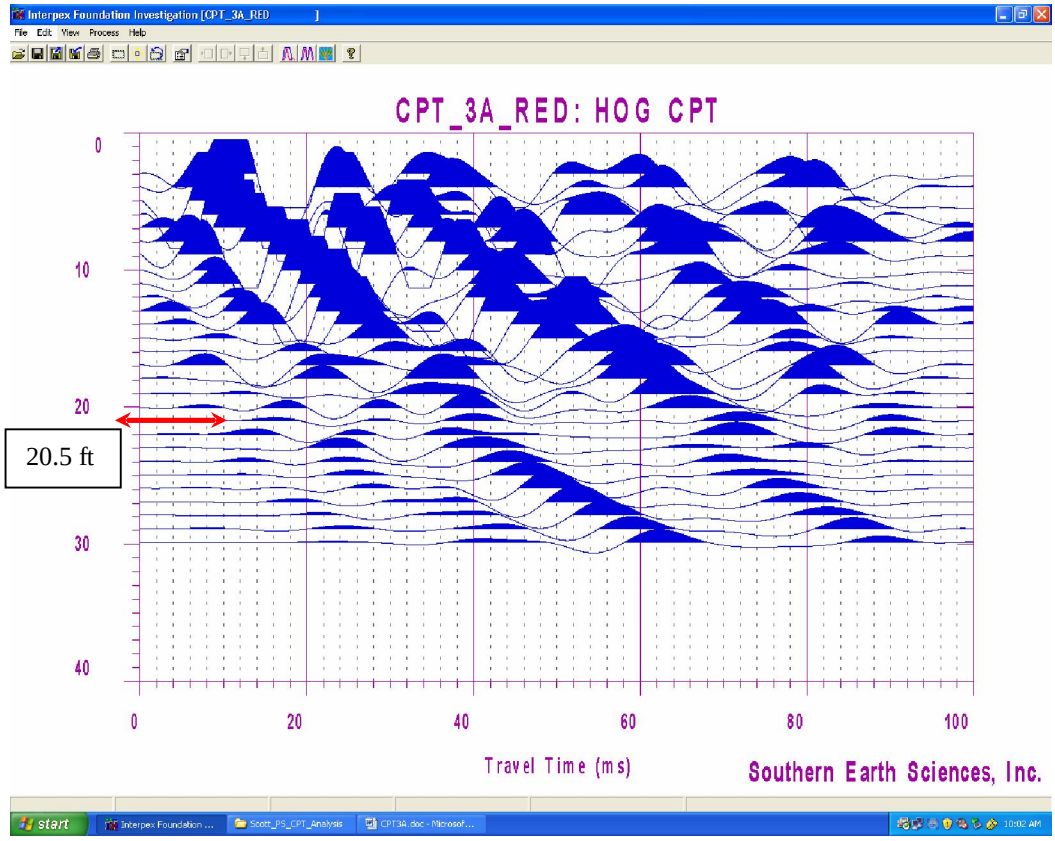
17th Street Canal Levee
PS/CPT Data Analysis
(From Previous Investigation)

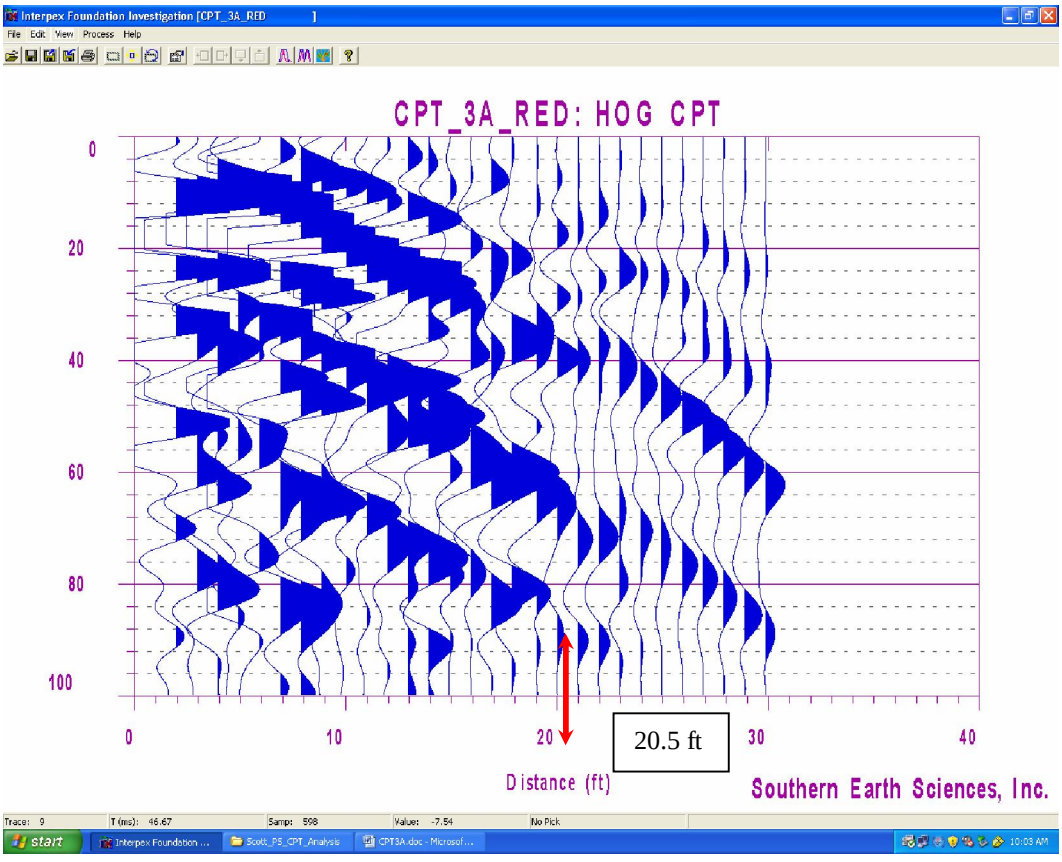




GPS Coordinates: N30°00.945' W90°07.289'; approximately 300 ft south of levee breach. Field-testing performed on Nov. 10, 2005.

17th Street Canal Levee
PS/CPT Data Analysis

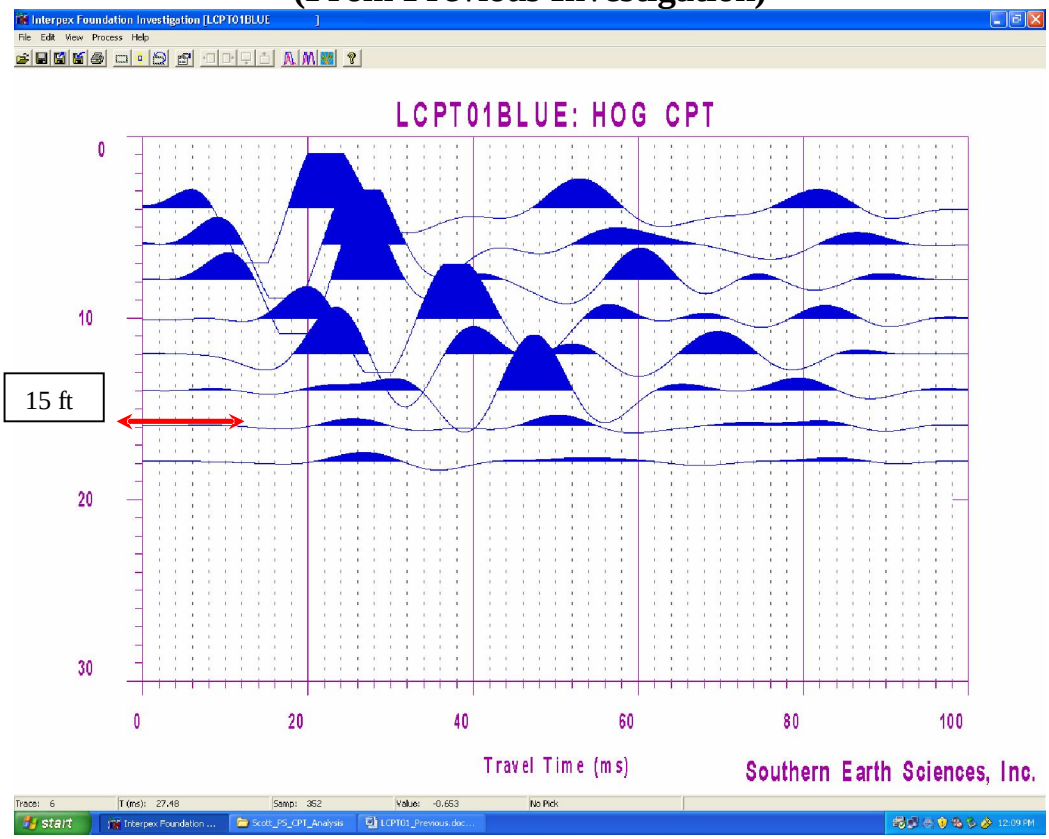


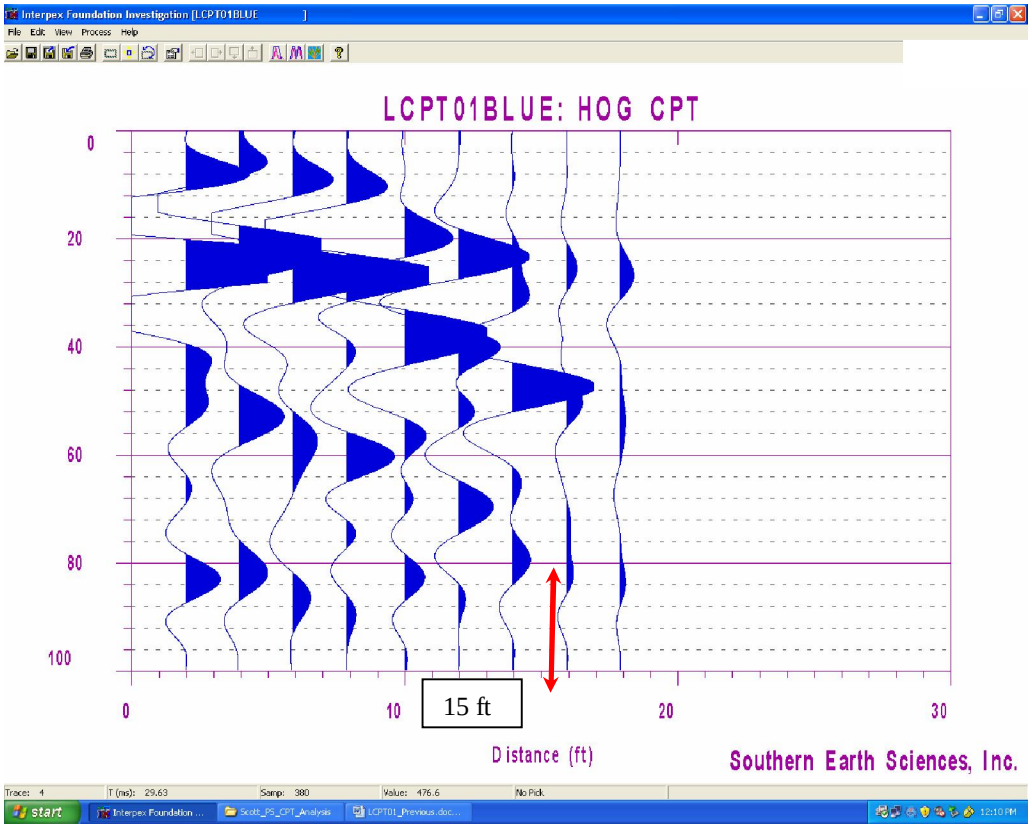


Sta. 4+08, approximately 200 ft south of breach. Field-testing performed on Dec. 22, 2005.

PS_LCPT_1

London Avenue Canal
PS/CPT Data Analysis
(From Previous Investigation)





GPS coordinates: N30°00.538' W90°04.167'; London Avenue Levee @ Maribeau. Field-testing performed on Nov. 11, 2005.